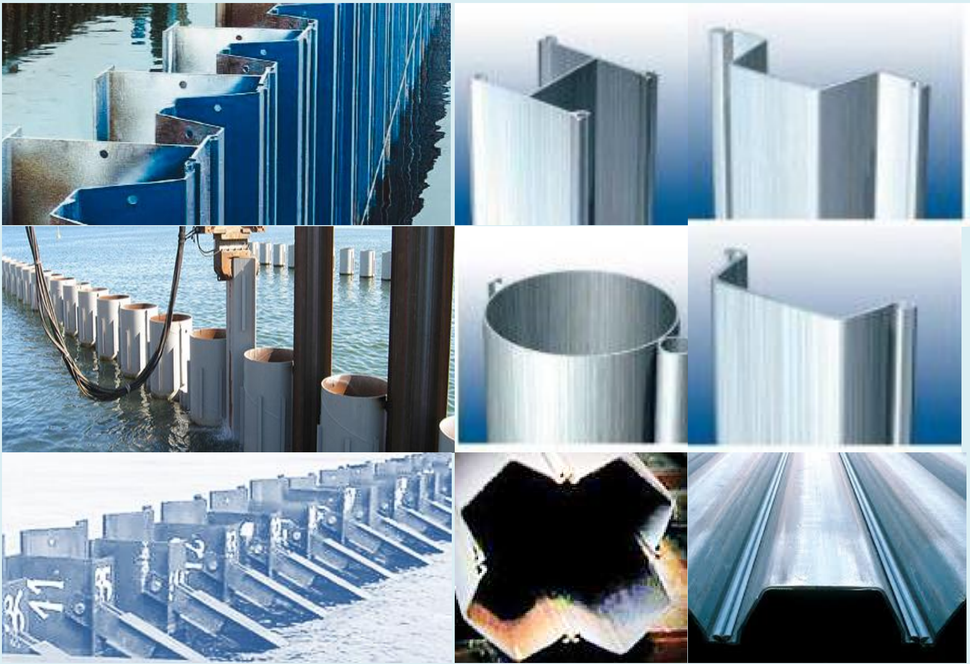
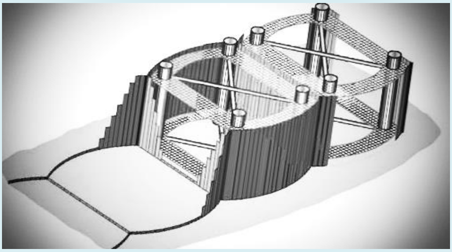
A Practical Approach to Design of SHEET PILE WALL STRUCTURES





MANOJ KUMAR

A Practical Approach to
Design of
STEEL SHEET PILE STRUCTURES

Manoj Kumar
Chief Consulting Civil Engineer
Grafix Engineering Consultants Pvt. Ltd.
New Delhi.

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Preface

This book is intended to be a reference manual of "Design of steel sheet pile structure" primarily for practicing engineers and anyone who is interested in the methods to apply the experience in real life situations. The design of sheet piles for a system is the application of scientific knowledge and engineering techniques adaptive to the physical properties of the ground. For an efficient design of deep embedded sheet pile wall, it requires very precise information of soil-mechanics, structural design verification procedures, and geometric properties of sheet pile elements which can be installed in to the ground with relative ease without compromising the safety or economy. The aim of this book is to present a simplified approach for design of sheet pile structures for various applications with an objective to familiarize with geotechnical aspects of relatively simpler problems, and provide a technical reference for complete discussions oriented to sophisticated analysis and design procedures. However, the book is not intended to be a textbook on soil mechanics and structural engineering but it does include very concise details of soil-structure interaction. The principles of soil mechanics and structural design verifications are stated only briefly, the reader should refer to the relevant textbooks for explanations of its theory.

Here a realistic computational model is more emphasised than an exaggerated numerical approach. A co-ordinated approach of statical calculation with computer aided design is presented to simplify the mechanisms for the design applying the various design loads and resistances. At the same time, various recommendations and codal provisions have also been streamlined and summarised as far as possible to improve the practical aspects. The design of deep embedded retaining wall needs a synchronised behaviour of earth retaining wall and anchorage system with surrounding soil environment. It needs a balancing act of wall design, earth work and stability of overall system. This book presents state-of-art information on basic principles and design analysis of various anchored system for retaining wall including the issues such as subsurface investigation and laboratory testing, basic anchoring principles and construction materials.

For preliminary design work, typical values of soil parameters are provided as a guide only for various situations to enable the engineer to proceed with the work related to geotechnical investigations, formulation of alternatives with various structural elements and to assess its implications on final design. Further, the contents in each chapter of the book are presented in a sequential fashion but more random approach is adopted for various design issues assuming that the reader has a grasp of fundamentals of soil, rock and structural mechanics. The book is primarily design oriented with more emphasis on the design of various elements of steel sheet pile wall system with minimum extent of theory presented wherever required for applications in design. Design examples are also provided at the end of each chapter for a better clarity on design issues.

While preparing this book, the author records sincere thanks to his colleague, Mr. Supratim Chanda for his cooperation and providing review comments to the text of the manual.

The book of this nature has to necessarily refer to published material from various sources, including those which are specifically mentioned in the epilogue. The author gratefully records his indebtedness to the sources in the technical literature and specifically the authors of the above references.

Finally the author welcomes constructive criticisms and suggestions from the professionals, engineers as well as teaching professionals/ scholars for further improvement and updating the contents of this book.

Manoj Kumar

mk.sheetpile@gmail.com

New Delhi

December, 2021

Chapter 1

Introduction

Engineering is not merely knowing and being knowledgeable, like a walking encyclopaedia; engineering is not merely analysis; engineering is not merely the possession of the capacity to get elegant solutions to non-existent engineering problems; engineering is practicing the art of the organizing forces of technological change ... Engineers operate at the interface between science and society.

- Gordon S. Brown

Use of sheet pile structure shall not be considered as a crude and brutal process. It is a versatile construction material possesses a number of unique features that make it an ideal solution for many applications in the construction industry. Steel provides unbeatable speed of construction and off-site fabrication, thereby reducing the risks associated with site-dependent delays. In particular, steel construction, with its high strength to weight ratio minimizes the self-weight and make it best suited option for offshore structures and marine construction. Steel sheet-piling is used all over the world for all types of structures, some of the more traditional applications are in construction of retaining walls, docks, jetties, trenches, and cofferdams. Steel has great advantage in pile driving as high strength of steel gives high resistance against impact, bending, lateral loading, and heavy compression.

However, inadequate sub-soil exploration and selection of inefficient sheet pile sections and equipment leads to chaos during construction. For an efficient design and execution of sheet piles, it requires a holistic approach with full understanding of the role of the wall in overall from planning stage to end of execution. It can be very precisely installed on land and water and even under the water condition. In the age of rapid urbanisation and changing industrial scenario, driven steel (sheet) piles provide a cost-effective solution for;

- Upgradation of existing dock existing dock and quay structures,
- Deep underground basement for major urban buildings
- Underpass and tunnel for roadways'
- Cofferdam for dewatering and work below the water level,
- Shoring and protective structure to mitigate the excavation hazard.

However, adequate geotechnical investigation, interpretation of data, establishing sub-soil profile, obstacles, uncertainties etc. are the key for efficient, economical design and successful installation of sheet piles. It is important to adopt a clear, unambiguous design method and to follow appropriate good practice of ground investigation, laboratory and field testing and design analysis, and utilise good-quality case study data.

1.1. Development in Steel Sheet Piles

Hot rolled sheet piles have been used for over a century and in due course of time they have undergone through many changes. When it first started, the sheets were flat. Eventually, they became U-shaped, and then transitioned to the Z shape. As a product it has been subjected to regular transformation in terms of shape, properties, configuration and installation methodology.

1.2. Development in Shapes

Now let us look into the development process and history of sheet pile sections. The original interlocks devised for steel pilings were rather crude, riveted additions to a basic structural shape. First versatile hot-rolled steel sheet pile section was developed in 1906 by Trygve Larssen, that greatly increased the strength and efficiency of steel walls. This Larssen shapes with integral interlocks revolutionized the sheet pile industry both as to the quality of the product and the cost.

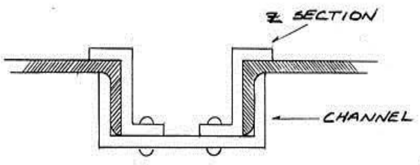


Figure 1.1(a) Primitive sheet pile section with riveted interlocks

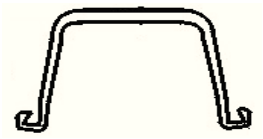


Figure 1.1(b) Initial hot-rolled U shaped sheet pile

In early 19th century, manufacturing Mills had limited capacity to roll the wider steel sections and initially developed sections were narrow U shaped piles. As the rolling capacities of mills were improved, wider and deeper pile sections were introduced to enhance the sectional properties of piles and strength to weight ratio too. This principal has influenced piling shape to ever larger and wider sections.

Later on Z piles were introduced in Europe, which further improved the sectional properties of sheet piles. Based on Larssen concept for a wave-shaped profile, interlocks of piles were formed on the outer elements (flange) of the section, which added following advantage;

- In U type sheet piles interlocks were formed on the Neutral Axis. In flexural members, neutral axis has the zero bending moment and maximum shear force, where sliding of interlocks is not desirable. At neutral axis position, continuity is required to transmit the shear force induced in the wall. Crimping and welding of interlocks prevent the relative slippage of wall, but this has to be over full length of wall which cannot be applied universally.

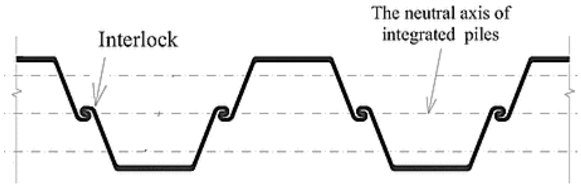


Figure 1.2 Arrangement of U-type Sheet Piles

- On the other hand, extra steel material provided to form the interlocks do not contribute to the bending strength since the lever arm of the interlock is zero, this is wasteful of the steel area in the interlocks. In Z-shaped piles interlocks are located on the flange side and the extra metals for interlocks add to the section modulus of pile.

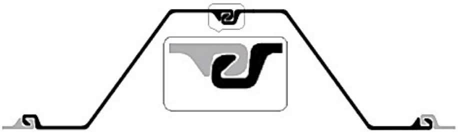


Figure 1.3 Arrangement of Z -type sheet pile

Today wider and deeper sheet piles are available. The width of U shape piles is up to 750mm wide and Z piles are up to 800mm. These piles offer maximised economy (optimised strength to weight ratio).

1.3. Development in Strength of Sheet Piles

As the shape and geometry of sheet piles continuously developed, the grade of steel used has also experienced a dramatic change. In 1960’s steel piles were rolled in grade 43A (with a yield of approximately 270 N/mm2). Later a “High Yield” grade was introduced in grade 50B (with a yield of approximately 350 N/mm2). In today’s market sheet piles of grade S270GP is quite rare and is generally not strong enough for normal requirements. The generally recognised “basis grade” for sheet piles is S355GP.

However, stronger grades are now generally available in S390GP, and S430GP, with yields of 390 N/mm2 and 430 N/mm N/mm2 respectively. In the next version of EN 10248 an even higher grade will be available in S460GP with 460 N/mm2 yield.

These higher grades bring several benefits. As the moment capacity of pile is directly proportional to the yield stress of material, higher grade steel requires lighter pile section and adds economy to the project. It is an effective solution to increase the grade of steel to optimise the total tonnage of sheet piles and easy to handle during installation.

1.4. Advantage of driven pile application

The application of steel sheet piles in retaining wall is highly advantageous. The advantages of steel sheet pile retaining walls are summarised below:

i) A driven pile is made of pre-formed material having a predetermined shape and size that can be physically inspected prior to and during installation.

ii) Driven piles are a total engineering solution. The design, installation and quality assurance that are a part of each driven pile combine to eliminate guesswork and produce a known, reliable and cost-effective product that can accommodate a wide variety of subsurface conditions.

iii) Sheet piles are pre-manufactured materials with precise tolerances utilizing high strength materials and reliable quality control.

iv) For driven piles quality is consistent from the first pile to the last and can be verified prior to installation.

v) A well-designed steel sheet piles, maintain their shape during installation. They do not bulge in soft soil conditions and are typically not susceptible to damage from the installation of subsequent piles.

vi) For steel sheet piles, various grades of materials and shapes are available which can be easily fabricated or specified for high structural strength and increased working loads, resulting in economical foundation Solution.

vii) Driven steel piles are installed to accommodate compression, tension and lateral loads to meet the specific needs of the structure, site conditions and budget.

viii) Driven pile installations usually produce no spoils for removal and disposal. The site is left clean and ready for the next construction activity.

ix) Driven piles adapt well to unique site conditions and restrictions. They are ideally suited for marine and other near shore applications and does not require special casings for offshore and riverfront installations.

x) Permanent sheet piling is a narrow form of construction, which can be installed close up to the boundary of the site maximising usable site space.

xi) Steel piles driven through water can be used immediately after installation, allowing construction to proceed in a timely manner.

xii) Driven steel piles require no curing time and Construction is significantly quicker than that for reinforced concrete walls.

xiii) Steel sheet piles are equally suitable for clayey and sandy type soil except in very hard strata where, special driving method is required.

xiv) The driving and extraction of steel sheet piles are easy and can be reused; therefore, it is equally suitable for permanent and temporary works. As they can be extracted easily, minimise the waste at site.

xv) A driven pile is a tested pile.

1.5. Contents of the Book

The main objective of this book is to provide the readers the deep insight about the planning, investigation, selection of structural configuration, design and planning for successful implementation of the structure. An understanding of the significance of several factors involved is required to be successful in the design of pile foundations. The designer must have a clear and through understanding of the loads, subsurface conditions including soil/rock properties and behaviour, the significance of special design events, foundation performance criteria, and current practices in structural design and construction in the area where the work is to be done to arrive at the optimum design basis.

While planning the steel sheet pile structure, the ground conditions may dictate the section properties of piles, and type of equipment needed to install the wall, which are likely to affect the cost. Obstructions and boulders can prevent the installation of sheet piles without pre-treatment of the ground to remove or break up the boulders.

Since, soil-mechanics is not a precise science because of inherent variability of the ground and the uncertainty of many factors associated with the construction. Adequate geotechnical investigation, establishing the ground profile and their successive properties is a pre-requisite for safe and economical and safe design of earth retaining structures and their timely execution. Therefore, the chapters associated in this book deal with creative planning, assessment of the geotechnical and structural stability and proper installation of sheet pile wall structure.

Chapter 2 provides an overview of sheet pile structure commonly used for construction. It provides basic details of sheet pile profiles, their configuration, combinations and applicability for structure.

Chapter 3 outlines about the material properties of sheet piles. Steel sheet is an international commodity and produced in various grades for different purpose. To meet the design requirement with economy, safety and logistics for transportation, selection of appropriate material grade is very important. Worldwide, it is manufactured according to various standards and to select an equivalent material corresponding to project standards, details are provided.

Chapter 4 A flexible retaining structures like sheet pile wall derives their basic loads and supports from the surrounding sub-soil strata. For safe and economical design of sheet pile wall and identify the potential of complexity for execution, obtaining more reasonable and accurate geotechnical information is very important. This chapter provides precise information about the planning of sub-soil investigation, scrutiny of field and laboratory data and assessment of suitable geotechnical parameters for design.

Chapter 5 describes about the necessary details to obtain the various static, transient and dynamic loads that may be applied externally to the structure. Methodologies for evaluating these loads are discussed in this chapter.

Chapter 6 is related to system stability of anchored sheet pile wall or bulkhead and methodology for design of various anchorage systems according to geotechnical condition of site, structural requirements and construction methodologies adopted for the project.

Chapter 7 outlines the procedure for stability analysis of structure to determine the response of a structure under applied loading conditions. For an adequate stability assessment, it is necessary to define "mode of failures" and to know the probable mechanism how it can fail. These failure modes and selection of certain system parameters to prevent failure are discussed in this chapter. To carry out analysis and design, engineer has to simplify and to idealise the structure according to real world existence. An idealised structure can be expressed with SSI (soil-structure-interaction) model by expressing the properties of soil in terms of subgrade modulus as describe in this chapter.

Chapter 8 covers the durability aspects for design of steel sheet piles. Steel sheet piles used in close proximity of marine environment or near saltwater are susceptible to corrosion due to higher concentrations of salt in the water, air and corrosion action becomes a design concern when the projected loss of steel thickness over the design life impacts the structural capacity of the piling. Outlines has been provided for the design of piles taking into account the loss of thickness or section properties due to corrosion or the suitable protective measure to counteract the corrosion effect.

Chapter 9 outlines the criteria for structural design of sheet pile wall. Here the structural design of sheet pile is based on Euro codes which involve different sets of calculations to determine the structural requirements of the wall to resist bending moments, shear forces and axial loads taking care of choice of suitable materials for use of the structure.

Chapter 10 outlines the ‘soil mechanics approach’ to calculate the load carrying capacity of driven piles under various conditions. The driving resistance and static load capacity of driven piles are potentially dependent on the behaviour of soil plug formation. In this chapter, the mechanism of driving resistance and formation of soil plug inside the piles are provided in detailed to provide the deep insight about the safe bearing capacity of piles

Chapter 11 is related to the drivability of pile which refers to the ability of a pile to be driven to a desired penetration depth. A pile will not be drivable into the ground, if driving system does not have drivability limit in excess of the soil resistance to be overcome during driving. The failure to adequately evaluate pile drivability is one of the most common deficiencies in driven pile design practice. In this chapter, comprehensive details of pile driving system involving the pile structure, the pile driving system/hammer, the soil and the other associated auxiliaries like crane, crane boom leads and the template are provided to assess the practical need of the pile installation.

Chapter 12 is about the creative application of sheet pile structures as it offers large number of configurations and solution to troubleshoot any retaining wall situation and scenarios or sub-soil conditions. In this chapter the application of sheet pile for waterfront structures are mainly discussed

.

Chapter 2

Sheet Pile Wall: General

Engineering refers to the practice of organizing the design, construction and operation of any artifice which transforms the physical world around us to meet some recognized need.

- GFC Rogers

2.1 Sheet Pile Wall

The term sheet piling refers to any retaining wall type that is;

a) installed into the ground by driving or pushing, rather than pouring or injection, and
b) made of relatively thin cross-section and low weight so that the weight of the wall does not assist in the wall’s stability.

The modern sheet piling industry is a little more than 100 years old with perhaps the most important changes in type and selection of products occurring since the early 1970’s.

2.2 Uses of Sheet pile walls

Sheet pile walls are retaining walls constructed to retain earth, water or any other filling materials. These walls are thinner in section compared to masonry walls. Sheet pile walls are generally used for following purposes:

i) Development of water front structures, i.e. in building wharfs, quays and piers.

ii) Building diversion dams, such as cofferdams

iii) River bank protection

iv) Retaining the sides of cuts made in earth

Sheet pile walls can be of timber, reinforced concrete or steel.

2.3 Material for Sheet pile

2.3.1 Timber sheet piles

Timber sheet piles are generally used for short spans and to resist light lateral loads. They are mostly used for temporary structures such as braced sheeting in cuts. When timber sheet piles are used in permanent structures above water level, they require preservative treatment and even, but still their life span is relatively short. Timber sheet piles are joined to each other by tongue and grove joints. They are not suitable for soils consisting of stones, as the stone would dislodge the joints.

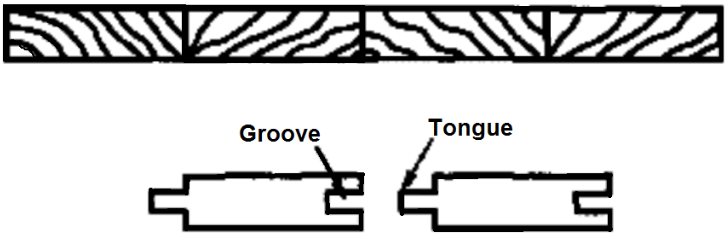


Figure 2.1 Configuration of timber sheet piles

2.3.2 Reinforced concrete sheet piles

Reinforced concrete sheet piles are precast concrete members, usually have a tongue and groove joints. Typical section of the RCC sheet pile is shown in figure below. These piles are relatively heavy and bulky. They displace large volumes of soil during driving. This large volume of soil displacement tends to increase the driving resistance. Due to this reason, adequate longitudinal and distribution reinforcement are provided considering large driving stresses.

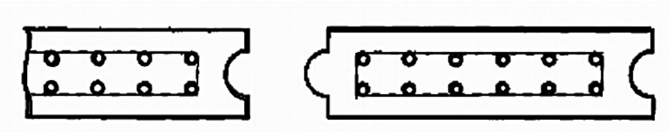


Figure 2.2: Reinforced concrete sheet piles

2.3.3 Composite Sheet-Pile Walls

Walls may be constructed using composite construction. The soldier beam-wood lagging combination is an example. Other examples include use of soldier beams on some spacing with sheet piling used between the spacing. For corrosion protection one might encase the upper part of steel sheet piling in concrete after it is driven, with the concrete extending from below the water line to the pile top. A wood facing might also be used, or the lower part of the sheeting could be made of steel and the upper part of a different material-wood or concrete (Bowles, 1996).
Since steel is relatively durable in most waterfront installations, the principal composite construction consists in using a mix of soldier beams and sheet piles or built-up box pile sections.

2.3.4 Fibre Reinforced Polymer

Sheet piling may also be manufactured of a synthetic fibre-reinforced polymer (FRP). This type of sheet pile is also referred to as fiberglass or composite sheet pile. An FRP product consists of fibre reinforcement and a polymer resin matrix. The fibre reinforcement typically consists of glass reinforcing fibres. Because of the method of manufacturing, the mechanical properties (strength, modulus of elasticity) may vary with orientation. Due to the potential strength of FRP sheet pile and its resistance to corrosion, FRP could be considered for applications requiring wall heights higher than allowed for vinyl sheet pile or for areas with high corrosion potential for steel. The required strength, modulus of elasticity, and anisotropic nature of the material must be considered in the design.

2.3.5 Vinyl Sheet pile

Vinyl sheet pile is available in sections of similar shape to Z-shaped steel sheet pile. Vinyl has lower strength and modulus of elasticity than steel and is, therefore, its applications are limited to lower wall heights or anchored walls. Vinyl sheet pile may be manufactured by mono-extrusion of all virgin material or co-extrusion of recycled material, coextruded with a virgin material coating to provide resistance to ultraviolet light.

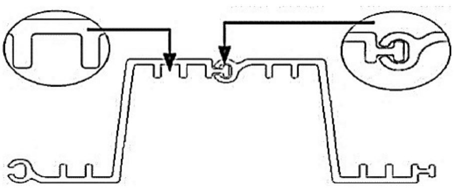


Figure 2.3 Vinyl sheet piles

2.3.6 Steel Sheet pile (Wall)

There are various types of earth-retaining structures. Where a differential surface level is to be established with a vertical interface, a retaining wall is used. Steel sheet piling is one form of retaining wall construction capable to resist the soil and water pressures by functioning as a beam spanning vertically between points of support. Steel sheet piles can be used for both temporary and permanent retaining walls. In temporary works, the sheet piles are used as cofferdam to enable deep excavations which are to be made below the ground and water level for other permanent works. On completion of construction the sheet piles are usually extracted for reuse on other places. Applications have seen the use of steel sheet piles as permanent retaining walls for structures including quay walls, bridge abutments, underground storage tanks, basements and underground structures.

Where retained heights are large, combined walls are used. These retaining walls comprise of high modulus piles and may require multiple propping at various positions. Steel sheet piles are most commonly used structural elements for embedded retaining wall. It possesses several advantages over piles of other materials. They are:

- Steel sheet piles are resistant to high driving stresses
- They are lighter in section
- They can be used several times
- They can be used either below or above water and have long life span
- Suitable joints can be provided to have a continuous wall
- The pile length can be increased either by welding or bolting.

Generally steel sheet piles are available in five profiles designated as Z, U, O, H and straight web. They are predominantly manufactured as hot-rolled section, although cold formed sheet piles are also manufactured except for straight web section. The details of various profiles are follows.

2.4 Types of Steel sheet Piles

2.4.1 Z type sheet piles

Z sheet pile refers to a single profile shaped like a stretched Z as shown in Figure-2.3. This profile is continuous through the web, and the interlocks of the pile are located symmetrically on both sides as far away from the neutral axis as possible. Both factors create a positive effect on the section modulus for the sheet pile. Z sheet piles are traditionally driven as crimped double piles.

The advantage of Z pile:

a) High moment of inertia to limit deflection
b) High bending moment resistance
c) Continuous web
d) high-cost efficiency result from high modulus section and relatively low weight

The single Z-pile is unsymmetrical in shape may experience twist during handling and driving, therefore, these are normally driven as in crimped or welded doubles. This type of profile is used for intermediate to deep wall construction as they are considered to be the most efficient type of sheet pile available. Z-piles are commonly used for cantilever and tied-back retaining wall systems.

2.4.2 U type sheet piles

U type sheet piles are in U profiles with interlock at both sides to form the continuous walls, which has centre lines in the middle of the double U-section wall. U-profile sheet piles are used for similar applications as Z-profiles, however, because the interlock between adjacent sheet piles is at the neutral axis of the section, their bending resistances are lower than that of comparable Z-profiles. However, it has larger section modulus.

Narrow U sheet piles are suitable for hard ground condition and wider U piles are also applicable for ground easy to drive.

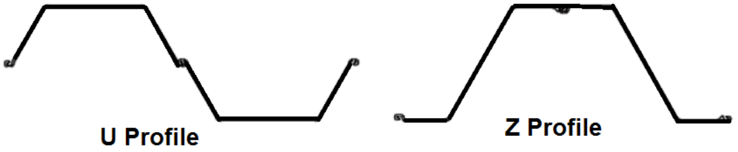


Figure 2.4 U type and Z type steel sheet piles

Compared to standard U profile sheets, Z sheet piles offer greater system width, providing reduced handling and installation time. Using Z piles also reduces the number of interlocks per meter run of a wall, offering enhanced water tightness.

2.4.3 Jagged Wall

Jagged walls are "special sheet pile wall configuration in which the single Z-piles are arranged either to enhance the moment of inertia of the wall or to suit special applications.

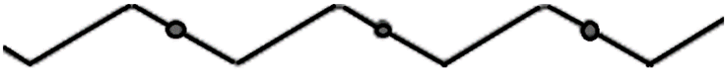


Figure 2.5 Jagged type steel sheet pile wall

2.4.4 Contiguous O-pile

An O-shaped sheet pile is a pipe section with attached connector sections so that one section can be driven into the next to form a continuous steel wall with the same load bearing element.

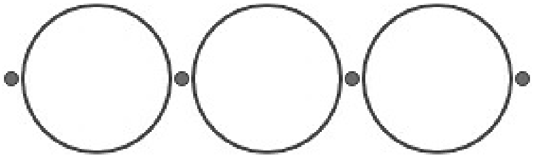


Figure 2.6 O- type steel sheet pile wall

2.4.5 Straight web sheet piles

Straight web sheet piles are used in construction of circular cells or diaphragm cells depending on the site characteristics or the design requirements. The sheet pile enclosure (Cells) contains soil fill to form a solid mass and its stability is ensured by action of gravity. Straight web sheet piles are designed to form cylindrical structures, generally closed, retaining a soil backfill and piles are subjected to tensile forces, and hence interlocking strength is of prime importance.

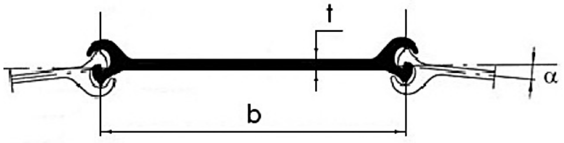


Figure 2.7 Straight web sheet pile wall

2.4.6 High Modulus Piles

Buttressing a sheet pile wall with deep universal beams placed in 'soldier' fashion creates a high modulus pile as shown in Figure-2.7 below. This type of section provides additional bending moment capacity and bearing capability. The UB's are welded back to a pair of Z profile sheet piles and all three are driven clutched to the next panel.

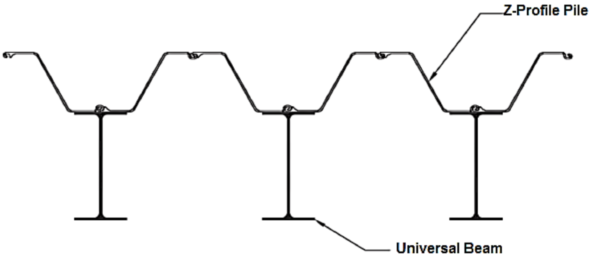


Figure 2.8 Z-type sheet pile wall + H-Beam System

2.4.7 Combined Walls

Combined walls are another form of part-fabricated piling used to form walls for deep retention of backfill or excavations works. They comprise steel bearing piles acting as structural supports in conjunction with secondary sheet piles acting as intermediate earth retaining infill elements. These walls provide increased stiffness or if necessary, provide improved bearing capacity.

The combined wall can be formed from alternating tubular, H and box sheet piles. The tubular piles take vertical loads and stiffen the wall. See Fig 2.9. For a HZ walls system, proprietary I sections are interlocked with Z profile piles, using a specifically designed hot-rolled connector. See Fig 2.8.

Also, for heavy marine constructions and deep foundation applications the piling wall often requires a very high bearing capacity and stiffness. Where a regular sheet pile wall is not sufficient, a combi wall system is used to achieve the required properties. Combined wall systems, often called combi-walls for short, are often employed when regular sheet piles are not strong enough to support the required load. It is a combination of king pile and intermediate infill sheet piles. The king pile provides the bulk of the wall's structure, while the sheet pile is used to fill in the spaces between, making the wall continuous. Functionally, king pile are rigid primary walls spaced at certain interval to bear the vertical and horizontal loads and infill sheet piles are secondary piles spans between the king piles acts as a diaphragm and transfer the horizontal loads on king piles. As king piles H-piles, pipe piles or box piles can be used and double-Z pile or combination of U piles are generally used as infill sheet piles. The king piles in a combi-wall system act as a rigid and strong frame and when installed vertically parallel to each other to retain rigidity of wall with other structural elements.

They offer increased wall stiffness and increased wall movement resistance. On the other hand, they're also a cost-effective shoring alternative to the secant wall, making them a more economical option for larger projects. Finally, king piles can extend deeper into competent bearing strata and the infill sheet piles can be driven to a limited depth to act as retaining diaphragms between the king piles.

2.4.8 Combined Sheet Piles: H + Z

H + Z Sheet Piles is a combined steel sheet piling wall system comprising H-sheet piles (as king piles) connected to Z-sheet piles by special connectors.

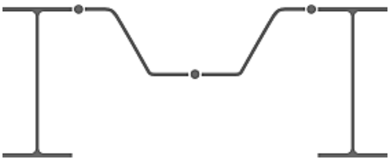


Figure 2.9 H+Z combi-wall

Connectors are the special steel sheet pile elements that connect adjacent king piles and sheet piles to make a continuous wall. The Interlocks shall have adequate free play, so that the piles can be fitted into each other and they must engage in such a manner that the in-service forces can be transmitted. The interlocks or connecting system in combined H + Z sheet piling also allows for a 4 mm minimum engagement distance in order to provide a free play between the sheet piles during installation and effectively transfer the forces when in use.

2.4.9 Combined Sheet Piles: O + Z

O-Pile + Z-Sheet Pile is a combined steel sheet piling wall system comprising O-shaped pipes or tubes, acting as the primary elements, connected to Z profiles via special sheet piling connecting sections.

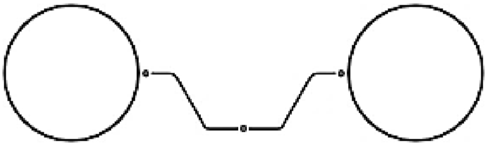
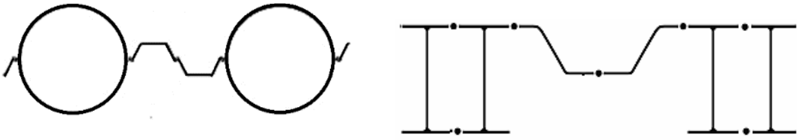


Figure 2.10 O+Z combi-wall

The others combined sheet pile systems are as shown in Fig 2.10.



O+U Combi-wall H Z Combi-wall

Figure 2.11 Other combi-walls

2.4.10 Sheet Piles and Box Piles

Steel sheet pile section walls are increasingly used for bearing pile applications in combined loading situations. Applications for permanent sheet pile walls in bearing are used in basements, underground structures. Box piles are formed by welding two or more sheet pile sections together. Both U and Z sheet pile sections can be used. They can be introduced into a line of sheet piling at any point where local heavy loads are to be applied in basement. They are clutched together with adjacent sheet piles and can be positioned in a sheet pile wall so that its appearance remains unaffected.

U sheet pile box piles are formed by welding together two sheet pile sections with continuous welds and Z sheet pile plated box piles are formed by continuously welding a plate to a pair of interlocked and intermittently welded sheet piles (see figure below).

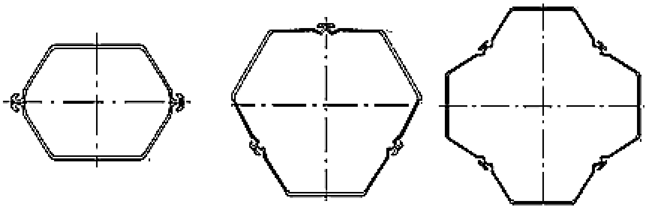


Figure 2.12 Box piles

Special box piles can be formed using other combinations of sheet piles.

2.5 Interlocks

Steel sheet piling is a manufactured construction product with an interlock connection at both ends of the section. These interlocks connect with one another to form a continuous wall of sheet piling. Steel sheet pile applications are designed to be interlocked together to create a rigid barrier for earth and water, while resisting the lateral pressures of those bending forces. The portion of a steel sheet pile or other sheeting that connects adjacent elements by means of a thumb and finger or similar configuration to make a continuous wall is known as interlock.

Interlocks may be described as:

- Free: Threaded interlocks that are neither crimped nor welded;
- Crimped: Interlocks of threaded single piles that have been mechanically connected by crimped points;
- Welded: Interlocks of threaded single piles that have been mechanically connected by continuous or intermittent welding.

Interlocks shall have adequate free play, so that the piles can be fitted into each other and they must engage in such a manner that the in-service forces can be transmitted. For non-flat sheet piling, BS EN 10248-2 also allows for a 4 mm minimum engagement distance in the interlocks in order to effectively transfer forces. Hot rolled sheet piling has minimum engagement distance as 4 mm while cold formed does not necessarily. This is why cold formed does not always transfer shear forces as effectively as hot rolled sheet piling. Interlock connections between hot rolled sheets provide much greater strength than cold formed connections. Greater interlock strength improves integrity during driving and allows forces to be redistributed laterally along the wall at changes in wall alignment.

The widely used interlocks in hot-rolled sheet piles are as given preceding sections;

2.5.1 Thumb and finger type Interlocks

The thumb and finger design provide three points of contact between the elements of the interlock when stressed. This not only accounts for the high strength of these connections, but also provides a double seal against water intrusion or soil leakage. It is widely used in flat web sheet piles.

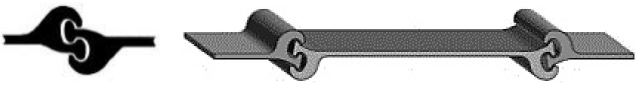


Figure 2.13 Thumb and finger type interlock

It is also designed to allow a degree of angularity or swing between interlocks. Traditionally, designers could count on at least 4 to 5 degrees swing per joint. However, it shall be get confirmed with manufacturer's catalogue. From this, the minimum diameter circle or arc that could be built using that particular sheet piling section could be determined.

2.5.2 Ball and socket Interlocks

The modern ball and socket interlocks are designed to “present a smooth, clean wall face, to locate metal where it would do the most good strengthwise, to present an efficient seal against leakage, to permit easy threading. If the ball of this interlock is driven leading, then it clears a path for the socket to follow.

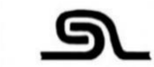
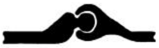
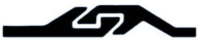
2.5.3 Jaw Type Interlocks

The Single Jaw interlock is similar to the ball and socket but with angular contours rather than oval. The Double Jaw design found on certain European and Japanese Z-type sheet piling is a unique design, with perhaps some additional advantages for wall corners.

2.5.4 Hook and Grip Type Interlock

Hook and Grip Interlock is the type utilized on all cold-finished sheet piles, both Z and arch types. This interlock is formed by bending the edges of the section during one of the last passes through the cold-forming rolls. Except for cellular cofferdams, most sheet piling is used for straight wall construction.

Accordingly, there is generally no need for swing guarantees between sheets. From a practical standpoint, some swing can be obtained and it provides the flexibility in wall construction to accommodate the driving tolerances. Interlocks must have sufficient clearance for sliding. Various interlocks for sheet pile section are shown in Figure 2.13.

	Hook and grip type interlock for Cold-form sheet pile section
	Ball and socket type interlock for Hot-rolled section
	Jaw type interlock

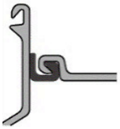
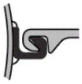
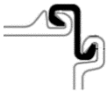
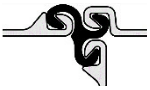
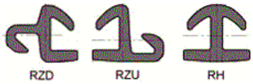
	C-9 type connectors for corner of sheet pile wall
	C-9 type connectors for Pipe pile and infill sheet pile connections
	C-14 type connectors for corner of sheet pile wall
	T- connectors for wall
	Connectors for HZ sheet pile wall

Figure 2.14 Interlocks and connectors for sheet piles

2.5.5 Flexibility in Interlocks

The sheet piling interlocks require some flexibity for practical reasons. Greater flexibility allows for easier driving in difficult soil conditions. A combined sheet piling wall systems flexibility usually allows for easy setting of the intermediate sheet piling between the already driven king piles (beams or pipes). The amount of swivel at each wall component interlock (measured in degrees or radians).

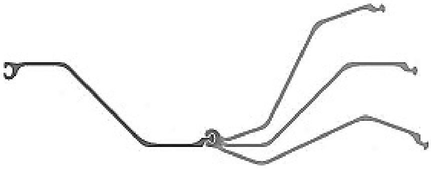


Figure 2.15 Flexibility at sheet pile interlock

2.5.6 Interlocking in pairs

Z sheet piles are normally supplied in pairs which save time in handling and pitching and double-Z piles are symmetrical about its axis. Single-Z piles are unsymmetrical sections and susceptible to damage by plastic deformation under self-weight during handling and driving.

2.5.7 Crimping and welding of the interlocks

Crimping or welding of double-Z piles is not necessary to guarantee the strength of the piled wall, but can be of benefit during handling and driving. Pairs of piles can be crimped or welded together if required. Normally 3 to 4 crimps per metre are sufficient but other configurations can be accommodated. Generally, each crimp is applied to provide an allowable shear resistance of 75 KN with less than 5 mm movement, which should be confirmed from manufacturer’s catalogue.

2.6 Hot-Rolled vs. Cold-Form sheet Piles

Hot-rolled sheet piles are profiled in a rolling process at high temperatures. It is manufactured in a single- or two-step process heating pre-rolled steel beam planks to such a high temperature that it allows it to be easily shaped and formed into sheet pile. The hot rolling process is typically used for production of standard sheet piles, including U and Z-sections. The interlocks on both sides of profile are precisely formed during the rolling.

Cold-rolled sheet piling is profiled from wide-plate steel and manufactured using a cold-deformation process. Cold-rolled sheet piling is available in steel thicknesses of 3 to 16 mm and is used in light to medium-heavy earth and water-retaining structures.

Cold-formed is manufactured in two steps. First, hot-rolled steel coil is manufactured and later it is run through a mill, which forms the coil into the steel sheet pile sections. All cold-formed interlocks are basically similar; generally, ‘Hook and Grip’ type. Cold-formed interlocks are looser, therefore, threading and driving of these sheet piles are easier, which is especially an advantage for driving and pulling of sheets in temporary works. This feature, speed up the installation and removal process of sheet piles in temporary constructions.

- The real difference between the hot-rolled and cold formed sheet pile is in the interlock. The hot rolled sheet piles vary from one manufacturer to the others; therefore, hot rolled sheet piles from two different manufactures will need a transition piece. On the other hand, cold formed sheet piles have similar sizes, so they will interlock

— much like a hook and grip — regardless of which company manufactures them.

- Hot rolled sheet piles are deemed to be more durable and provide better stability for hard driving. The tight interlocks are made for tough conditions, so it will be able to endure, but require a well-designed template for positioning and aligning of sheet pies during driving. To compare, cold formed sheet piles are cheaper to make, more flexible to use and easier to source. They also give more choices for positioning or aligning the sheet piles
- Tight interlocks of hot-rolled sections provide a soil tight and water tight barrier to suit for permanent retaining wall, whereas cold-formed sheets are used for temporary cofferdams, trenching and shoring works.

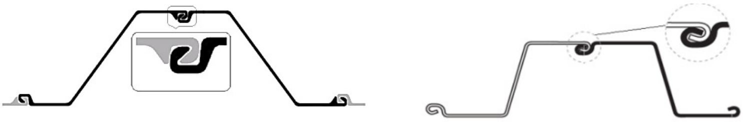


Figure 2.16 Hot-rolled sheet pile vs. Cold-form sheet pile

2.7 Common Applications of Steel Piling Products

The advantage of sheet piling is that it is a pre-manufactured product with known properties, which can be shipped to a project site and assembled, under diversified conditions, into a permanent wall. Sheet piling has a potential application anywhere a retaining or membrane type wall is needed. The application may be either temporary or permanent. In general, it has been used mostly for bulkheads at marine terminal facilities, marinas, and waterfront structures. It has also been extensively used for navigation structures, water control, floodwalls and land walls.

In following sections traditional ways in which the steel sheet pile products have been applied are described for number of innovative uses.

2.7.1 Marine Bulkheads

Sheet piling can be used a number of ways in the design and construction of a port operating facility. An anchored sheet pile bulkhead provides a vertical wall, behind which useable land area may be created and in front of which ships may be tied up for loading and unloading. Anchored bulkheads obtain their stability from the support provided by penetration into the soil below the dredged bottom combined with an anchorage support system installed near the top of the wall.

The sheet piling is then designed as a beam on two supports under the loads from soil pressure, water pressure and surcharges. The bending moment in the beam must be countered by the shape and tensile strength of the sheet pile “beam”. For most deep applications, anchored bulkheads utilize Z, U, H-Z or O-Z type sheet piling system.

Anchored bulkheads may be located parallel to shore (marginal wall) or perpendicular to shore (finger pier). It may be built for planned dredged depths of up to about 20 meters of water by utilizing the strongest sections currently available, combined with high strength grades of steel. Sheet pile walls are also utilized in the construction of deck-type wharves, as a screen (retaining) wall between the rear of the deck and the land area.

Special high modulus sheet pile wall systems have become a factor in deep-water marine facilities construction. This method utilizes large “king piles” as the main strength units of the wall, which possess strengths several times that available with conventional sheet piles.

2.7.2 Cellular Bulkheads

Bulkheads for special situations can be created from cellular sheet pile structures.
These applications include sites

i. where high rock prohibits the required penetration for support of a single wall,
ii. where deep water and heavy loads make a single wall unfeasible,
iii. where tidal fluctuations are extreme,
iv. Where site development is to take place over an extended period of time or combinations of these situations.

Cellular structures are extremely useful for land creation and development projects. The cells are self-supporting and can be built in advance of landfill materials. They also provide a curtain to prevent water pollution and, as vertical structures, take up much less space than a dike. Cellular bulkheads may be of either circular or diaphragm construction as shown below. Circular cells are more popular for stability reasons, although the diaphragm type can utilize a lighter weight pile and provides a somewhat flatter face for breasting vessels.

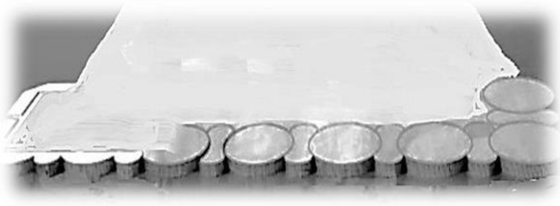


Figure 2.17 Typical Cellular Bulkhead arrangement for Jetty and backup area

2.7.3 Barge Docks

Self-supporting cells are utilized for other port operations such as mooring and turning dolphins. Single, circular cells in series have been used along the inland rivers for mooring fleets of barges. These are built sufficiently high to accommodate fluctuations in water levels. Some facilities have as many cells spread over a mile of shoreline to accommodate barge fleets.

2.7.4 Basements

Sheet piling is an ideal material for basements as minimal site width is lost maximising useable space. Piles can also be driven rapidly in the majority of soil types and are capable of being loaded immediately, which is a distinct advantage in fast-track construction projects.

2.7.5 Bearing Piles

Steel bearing piles can take the form of tubes or box sections made up from readily available Z or U sections and can dramatically decrease the construction programme as once installed they can be loaded immediately. They also have the added advantage of good sustainability credentials (piles can be extracted after use and reused) and due to the method of installation each pile is effectively load tested giving confidence in the underlying ground conditions.

2.7.6 Retaining Walls

Continuous retaining walls can quickly be constructed using steel sheet pies, either for permanent or temporary applications. The walls are most often used to retain either soil or water by transferring pressure from the high side of the wall to the soil in front of the wall. Use of high modulus rigid piles can also enable the wall to take high vertical loads as often used for quay walls.

2.7.7 Protective Structures

Breakwaters are built to protect the facilities and marine traffic in harbours. Large cellular cofferdam structures have been built to create artificial harbours. These cells are generally filled with granular soil, crushed stone and topped with armour stone or concrete. Stone berms may be added on both sides for further stability. Sloped face with rubble armouring has also employed in order to intercept breaking waves at a more favourable angle and reduce the reflection. The other special applications of sheet piles for marine facilities are as follows;

- **Seawalls** are structures built parallel to the shore for the purpose of protecting onshore properties from storm activated wave action. Since these structures are designed to trap and break waves. Sheet pilings are also employed as cut-off walls in and under stone or concrete walls to prevent under-scour.
- **Breakwaters** at marinas or small boat harbours have used sheet piling in the form of filled cells, single-wall, braced construction, or as a cantilever wall supported by a rubble-mound dike.

2.7.8 Artificial Islands

Large-diameter artificial islands can be created from filled sheet pile cells for locating transmission towers in bodies of deep water, for oil drilling platforms, etc. The retained soil takes up less space than sloped fill, is protected from scour and the steel cells offer some protection from ship collision.

2.7.9 Steel sheet Piles for Temporary Applications

While no statistics are kept, it is estimated that at least half of the annual production of sheet piling is utilized for temporary applications. Steel sheet piling, particularly, has the inherent strength to allow it to be driven, pulled, and reused a number of times before its value as a tool is lost. In following sections application of sheet piles for temporary applications are described for number of uses.

2.7.9.1 Temporary Box Cofferdams

Single-wall box cofferdams are the means by which bridge piers and other heavy foundations with relatively small area are constructed under dry conditions. Sheet piling sections form a structural wall that is generally braced internally against the soil or water pressure outside. The sheet pile membrane excludes earth and water from the site while the permanent work is being constructed. Since these piling sections are subject to bending stresses, “Z”, arch or “U” type sheet piling sections are utilized.

2.7.9.2 Land Cofferdams

Steel sheet piling may be considered for retaining walls of any temporary excavation. While other methods for retaining soil such as soldier beams and lagging, slurry walls, soil nailing etc. have evolved, sheet piling is advantageous where high ground water or flowing soil loss threatens not only the new excavation but also adjacent existing foundations. Sheet piling can be driven below the proposed excavation level in order to cut off flow into the bottom of the hole. Raker beam bracing or strutting are traditional means of supporting large sheeted excavations. This can also be accomplished with external tiebacks into soil or rock. A variation of the rectangular or square box cofferdam is the circular type. This design employs circular compression ring walers rather than cross bracing and frees the inside to allow more efficient digging and placement. If the diameter is sufficiently large, the circle may be closed by the natural swing between sheet piling sections, otherwise, a number of pre-bent sheets must be provided for in the design.

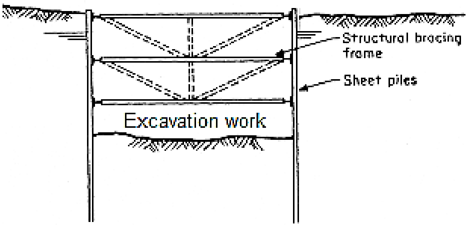


Figure 2.18 Land Cofferdam for excavation

2.7.9.3 Trench Excavations

It has become almost mandatory that major trench excavations be sheeted and shored due to following reasons:

a) To meet local municipal rules;
b) To protect the workmen;
c) To protect adjacent structures; and
d) To keep the excavation area free of earth and water.

Most trenches required relatively shallow excavations, lighter-weight sheet piles may be satisfactory and there are a large number of shapes and sections from which to choose for this purpose. Trench excavations require bracing consisting of one or several rows of horizontal walers with cross bracing, spaced to permit machine-excavation and installation of the permanent construction. Some bracing may be prefabricated and also serves as a guide template for aligning the piling.

Other temporary applications for straight walls include:

a) Temporary support of embankments or existing foundations on highway or subway construction projects.
b) Construction of temporary bin walls for separating materials in bulk storage yards at job-sites.
c) Temporary marine facilities such as a receiving dock to allow offloading of heavy equipment.

2.7.9.4 Cellular Cofferdams

A cellular cofferdam is a gravity retaining structure formed from a series of interconnected cells filled with selected soil.

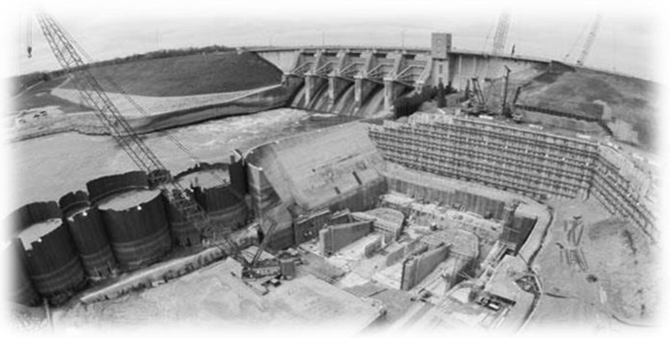


Figure 2.19 Cellular cofferdams as water barrier between the river and powerhouse excavation

Cellular cofferdams can be used for temporarily excluding water from deep, large excavations such as for locks, dams, and hydraulic structures, and also for permanent docks or walls where single wall bulkheads cannot be used.

2.7.10 Highway Applications

2.7.10.1 Retaining Walls and Abutments

In Europe, sheet piling is regularly employed for fill retention on highway ramps and underpasses and along cuts and embankments. This system takes advantage of the ability of steel shapes to carry vertical loads as well as horizontal loads.

2.7.10.2 Bridge Pier Protection Cells

In navigable waterway area, where vessel collision by movement of ships and barge may be anticipated, existing bridge piers, foundations shall be protected to prevent collapse of the superstructure. This requirement applies to all bridge types which cross a navigable inland waterway or canal with bridge traffic waterways with merchant ships. Providing protection to existing bridges presents design, construction and economic Challenges. The practice of locating large sheet pile cells around critical main piers of major bridges to protect them from ship collision is more apparent. The cells are sacrificial and are designed to stop or divert the ship so that any impact will not affect the stability of the pier and the main structure.

2.7.11 Other Applications

Sheet piling should be considered for any application where a vertically faced wall is required.
Some of the innovative uses that have been made are:

a) Curtain wall surrounding shallow water oil well drilling and production sites, to retain spillage and protect the environment.

b) A similar application around large, circular petroleum storage tanks at a refinery.

c) Scrap pits built in metal working mills utilizing a box-type structure built of sheet piling. Concrete deteriorates quickly from abrasion and impact of the heavy scrap.

d) Separator walls for bulk material storage.

e) River control structures and flood defence -Steel sheet piling has traditionally been used for the support and protection of river banks, and flood protection. Ease of use and the ability to be driven through water make steel sheet piles the obvious choice.

Chapter 3

Material Properties

Each material has its specific characteristics which we must understand if we want to use it. This is no less true of steel ....

- Ludwig Mies van der Rohe

3.1 Properties of Steel

Steel materials are international commodities which are commonly shipped thousands of kilometres from where they are manufactured to wherever there is a site for installation and use. It is used for a wide range of construction projects including port and harbours, river jetties, earth retaining system and cofferdam due to exceptional quality and work efficiency. Among various international steel materials specifications, European steel materials specifications have been widely adopted in many countries all over the world owing to their applicability levels.

This chapter presents essential technical details for material properties of driven steel sheet piles and selection of suitable materials conforming to European standards in respect of both material performance and quality check. However, for steel pipe piles and sheet piles, the equivalent steel materials conforming to API 5L, ASTM JIS, AS and GB Standards are also conferred.

3.2 Effects of Composition on material performance

In general, the chemical composition is the most important factor affecting the mechanical properties of steel materials. Adding chemical elements such as Carbon, Manganese, Chromium and Vanadium either during tapping or secondary steel making will increase the strengths of the steel materials. However, these additions not only add to the cost of the steel materials, but also adversely affect other mechanical properties such as ductility, toughness, and weldability. The chemical composition for each grade of steel material has to be carefully considered to achieve the required properties.

The effects of some commonly used chemical elements on the mechanical and material performance of steel materials are described as follows:

a) Iron (Fe)

Iron is the single most important element in a steel material, comprising roughly 95% of the steel material. Any steel material with a percentage of iron lower than 95% will not be classified as "structural".

b) Carbon (C)

Carbon is the second most important strengthening (hardening) element. Increase in amounts of carbon cause a decrease in ductility, toughness and weldability. Typical Carbon contents in modern steel materials range from 0.05 to 0.25 %.

c) Manganese (Mn)

An important element, Manganese also increases strength and hardness of the steel material, but to a lesser degree than Carbon. Increasing the Manganese content also decreases ductility and weldability, but again, to a lesser extent than Carbon. Typical Manganese contents in modern steel materials range from 0.50 to 1.70 %.

d) Sulphur (S)

Sulphur has detrimental effects on strength, transverse ductility, toughness as well as the weldability of steel materials. It also promotes segregation in the matrix of the steel material. For these reasons, it is generally considered an undesirable element, and the content of Sulphur is thus restricted to no more than 0.04 to 0.05 %.

e) Phosphorus (P)

As for Sulphur, Phosphorus promotes segregation in the matrix of the steel material. Increasing the Phosphorus contents increases strength and hardness, but reduces ductility and toughness in the as-rolled condition.

f) Chromium (Cr)

Chromium is primarily used to increase the corrosion resistance of a steel material. It also increases hardenability, strengths at high temperatures and improves abrasion resistance. Different types of weathering steel have various Chromium contents ranging from 0.1 to 0.9 %.

g) Copper (Cu)

Copper is the other primary corrosion-resistant element used in a steel material. It is typically at not less than 0.2 % for steel materials manufactured with the use of an electric arc furnace (EAF), and about 0.02 to 0.03 % for steel materials manufactured with the use of a basic oxygen furnace (BOF).

h) Silicon (Si)

Silicon is one of the most important steel material de-oxidizers, and by this means, it is very effective in removing oxygen from the steel material during the pouring and solidification process. Typical Silicon content of a steel material is less than 0.4 %, but it must be at least 0.1 % if the steel material is to be considered fully killed.

3.3 Effects of Chemical Compositions on Weldability

The most common measure of weldability is the carbon equivalent value, CEV, which is used to assess the combined effect of carbon and the other chemical elements on the cracking susceptibility of the steel materials. Based on testing of the steel materials within certain ranges of chemical compositions, various empirical formulas for determination of the CEV of steel materials are available:

$$CEV = C + (Mn + Si) / 6 + (Cr + Mo + Cb + V) / 5 + (Ni + Cu) / 15 \qquad (3.1)$$

3.4 Designation of Steel Grades

The designation of steel grades is defined in the product standard for hot rolled products and structural steels in EN10027-1. The classification of steel grades is based on the minimum specified yield strength at ambient temperature together with various parameters for other mechanical properties and delivery conditions as follows:

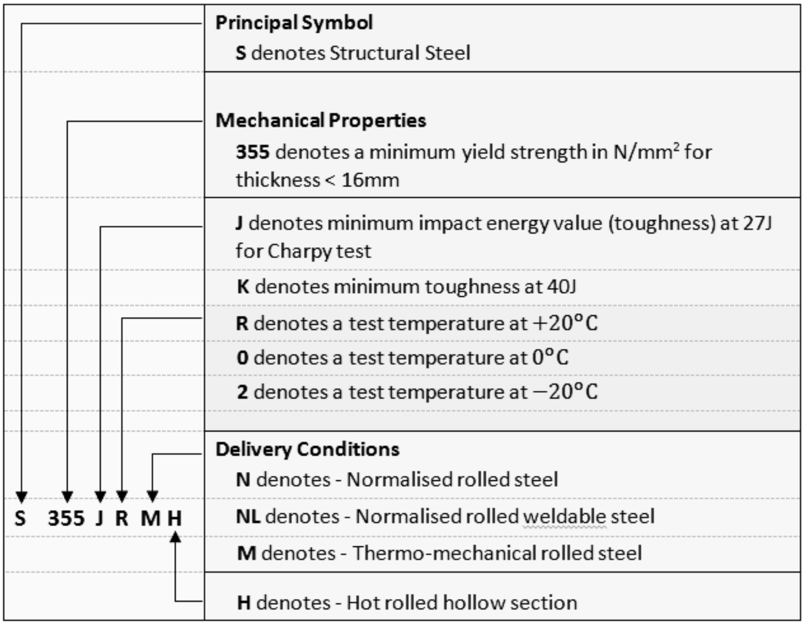

Principal Symbol
S denotes Structural Steel
Mechanical Properties
355 denotes a minimum yield strength in N/mm² for thickness < 16mm
J denotes minimum impact energy value (toughness) at 27J for Charpy test
K denotes minimum toughness at 40J
R denotes a test temperature at +20°C
0 denotes a test temperature at 0°C
2 denotes a test temperature at −20°C
Delivery Conditions
N denotes - Normalised rolled steel
NL denotes - Normalised rolled weldable steel
M denotes - Thermo-mechanical rolled steel
H denotes - Hot rolled hollow section
S 355 J R M H

Figure 3.1 Designation of steel grade according to EN10027-1

Further the grades of hot rolled steel sheet pile is denoted as S355GP, S430GP, etc.; where, the demarcation

S - denotes structural steel.

355, 430 - denotes minimum yield strength in N/mm^2

G - denotes for steel casting

P - denotes for sheet piling

The grade of cold form steel sheet piles supplied according to EN 10249 represented as S275 JRC, S355 J0C etc. Where,

S - denotes structural steel

275, 355 - denotes minimum yield strength in N/mm^2

JO or JR - minimum impact energy value of 27 J at 0°C for Charpy testing, others are JR (27 J at 20°CP - denotes for sheet piling

C - denotes cold form section

3.5 Basic Material Properties of Steel Materials

The basic material properties of steel materials are as presented below:

Material Property	Value
Density ρ	≈ 7850 kg/m^3
Unit weight γ	≈ 78.5 kN/m^3
Modulus of elasticity E (Young's modulus)	210000 MPa
Shear modulus G	$G = E / [2 \cdot (1 + v)] \approx$ 81000 MPa
Yield strength f_y	see Table 3.1
Ultimate strength f_u	see Table 3.1
Poisson's ratio in elastic range v	0.30
Coefficient of linear thermal expansion α	12×10^{-6} °K^{-1}

3.5.1 Unit weight and density of structural steel

The unit weight of structural steel is specified in the design standard *EN 1991-1-1 Table A.4* between 77.0 kN/m^3 and 78.5 kN/m^3. For structural design it is standard practice to consider the unit weight of structural steel equal to γ = 78.5 kN/m^3 and the density of structural steel approximately ρ = 7850 kg/m^3.

3.5.2 Modulus of elasticity of structural steel

The modulus of elasticity (Young's modulus) of structural steel is specified in the design standard *EN 1993-1-1 Section 3.2.6*. For structural design the modulus of elasticity of structural steel is considered as E = 210000 MPa.

3.5.3 Mechanical properties for structural steel

According to *EN1993-1-1 §3.2.6*, the design values of the material properties and coefficients for structural steel are:

Modulus of elasticity (Young's modulus): E = 210000 MPa

Shear modulus: $G = E / [2 \cdot (1 + v)]$ = 80769 MPa ≈ 81000 MPa

Poisson's ratio in elastic range: v = 0.30

Coefficient of linear thermal expansion: $\alpha = 12 \times 10^{-6}$ °K^{-1}

According to *EN1991-1-3 §3.2.6*, for calculating the structural effects of unequal temperatures in composite concrete-steel structures to EN-1994 the coefficient of linear thermal expansion may be taken as $\alpha = 10 \times 10^{-6}$ °K^{-1} i.e. the same as the coefficient of thermal expansion of concrete.

3.6 Selection principles of Steel Materials

Steel piles or sheet piles are primarily subject to axial (compressive and tensile) forces, bending moments and shear forces. They respond linearly to applied stresses up to a 'yield point', and thereafter exhibit a significant capacity for plastic straining after first yield. Alternatively, especially for high strength steel materials, they respond non-linearly to applied stresses, and the proof strength is considered to be achieved when a 'proof strain' at 0.2 % is attained. At large deformations, i.e. strains larger than 2 %, post-yielding strengths are significantly larger than yield strengths because of the effect of strain hardening, up to tensile strengths, typically, 15 to 25% in excess of the yield values.

The steel materials fracture eventually at a typical elongation with a strain limit of 15 to 25%. The key parameters that define the material performance of steel have been identified as follows:

a) Mechanical strengths for structural adequacy,

b) Ductility for sustained resistances at large deformations,

c) Toughness in terms of energy absorption against impact and driving stresses,

d) Chemical compositions and weldability for minimized risks of crack formation in welds.

a) Material strengths for structural adequacy

Both the minimum yield strength and the ultimate tensile strength of the proposed steel materials should be directly adopted from their national materials specifications. The nominal yield strength should be in the range of 235 N/mm^2 to 690 N/mm^2. The nominal tensile strength should be in the range of 300 N/mm^2 to 1000 N/mm^2.

b) Ductility for sustained resistances at large deformations:

Ductility of steel materials correlates approximately with their elongation limits, that is, the elongations of steel coupons at fracture in standard coupon tests. The values of the elongation limits depend heavily on the dimensions of the steel coupons and the testing procedures as well as the product forms of the proposed steel materials and the steel coupon sampling methods.

The elongation after fracture on proportional gauge length shall be at least 15 %, for nominal yield strength not greater than 460 N/mm^2; and shall be at least 10 % for nominal yield strength greater than 460 N/mm^2. The tensile strength to yield strength ratio shall be at least 1.2 based on nominal values, or at least 1.1 based on actual values, for nominal yield strength not greater than 460 N/mm^2.

c) Toughness in terms of energy absorption against impact and driving stresses

Toughness is an important mechanical property of steel materials, which is the resistance against brittle fracture, and is quantified as the amount of dissipated energy obtained from standard Charpy V-notch impact tests at various design temperatures. As a minimum, the product shall be able to absorb at least 27 J of impact energy at 20 ^{o}C.

d) Chemical compositions and weldability to minimize risks of crack formation in welds

The contents of number of chemicals in steel should be kept at an optimal limit, such as Carbon, Sulphur and Phosphorus as their presence tend to reduce ductility, toughness and weldability as well as promote segregation at the same time. As a simple rule for hot-rolled steel sections, the maximum Carbon content should not exceed 0.25 % while the maximum Sulphur content should not exceed 0.05 %. Moreover, the maximum Phosphorus content should not exceed 0.05 %, which is further limited to 0.01 % when a through thickness quality, i.e. Z quality, is specified.

The weldability of steel materials depends on the carbon equivalent value, CEV, which represents the combined effects of Carbon and other chemical elements on the cracking susceptibility of the steel materials.

3.7 Classification of Steel Materials

Based on satisfactory demonstration of both the material performance and the quality assurance during their manufacturing processes, steel materials with yield strengths from 235 to 690 N/mm2 are classified as follows:

Table 3.1: Chemical composition requirements for steel based on ladle analysis [Maximum content (% by mass)]

Steel grade EN 10248	**Min. yield strength ReH**	**Min. tensile strength R_m**	**Min. elongation $Lo= 5.65\sqrt{So}$**	**Chemical composition (% max)**					
	Mpa	**Mpa**	**%**	C	Mn	Si	P	S	N
S 240 GP	240	340	26	0.25	-	-	0.055	0.055	0.011
S 270 GP	270	410	24	0.27	-	-	0.055	0.055	0.011
S 320 GP	320	440	23	0.27	1.70	0.60	0.055	0.055	0.011
S 355 GP	355	480	22	0.27	1.70	0.60	0.055	0.055	0.011
S 390 GP	390	490	20	0.27	1.70	0.60	0.055	0.055	0.011
S 430 GP	430	510	19	0.27	1.70	0.60	0.055	0.055	0.011

Table 3.2: Equivalent steel grades for sheet piles according to ASTM, CSA and JIS

Europe	EN 10248-1	S 270 GP	S 320 GP	S 355 GP	S 390 GP	S 430 GP
USA	ASTM	A 328	-	A 572 Gr. 50; A 690	A 572 Gr. 55	A 572 Gr. 60
Canada	CSA	Gr. 260 W	Gr. 300 W	Gr. 350 W	Gr. 400 W	-
Japan	JIS	SY 295	-	-	SY 390	-

3.8 Common Steel Grades Used For Tubular Piling

3.8.1 Tubular Piles

Tubular piles or tubular pipes used for piling purposes, which are also called steel piles, pipe piles, or tubular piling. Tubular piles are normally large diameter piles commonly used for deep foundations because its capacity to bear large bending moments along with axial forces. Tubular pilings for combi-wall are available in numerous of grades and are also supplied with C9 or C14 connectors. Tubular piles are produced from different steel grade for wide applications e.g. pile foundation, king pile, mono-pile, bearing pile, pipe pile wall, deep foundation and deep-water dock pile or offshore structures etc. Piling tubes are manufactured in a range of materials, from standard carbon steel to high tensile steel, and in all thicknesses to suit specified design.

They are manufactured using seamless, longitudinal welded (LSAW), or spiral welded (SSAW) technic for supporting and designing the foundation of a structure.

The various specifications for pipe piles are as follows;

Table 3.3: List of Codes for Pipe Pile Materials

EN 10210 - 2	Hot finished structural hollow sections of non-alloy and fine grain structural steels
EN 10219 -1	Cold formed welded structural hollow sections of non-alloy and fine grain steels
API 5L PSL1	Specification for Line Pipe Gr. B X42, X46, X52, X56, X60, X65
API 2B	Specification for the Fabrication of Structural Steel Pipe (as an alternative to API 5L for tubular pile foundation)
ASTM A252	Specification for the Fabrication of Structural Steel Pipe
ASTM A500	Standard Specification for Cold-Formed Welded and Seamless Carbon Steel Structural Tubing in Rounds and Shapes
JIS G 3444	Carbon Steel Tubes for General Structural Purpose STK400, STK490, STK500 and STK540
JIS A5525	Steel Pipe Pile- SPS400, SPS490, SP500, SPS540

The commonly used specifications for seamless or welded pipe piles are as follows;

a) API 5L

This grade of pipe was developed for use in transportation pipelines for the natural gas and petroleum industries. Although API 5L is an excellent product, it is typically not designed for structural reasons when used in casing and pipe piling projects. It requires extensive testing for that purpose including hydrostatic testing, non-destructive, and destructive testing. API 5L products come in many grades, denoted by “X45” or “X65,” which refers to the yield strength (e.g., X65 has a yield strength of 65 ksi or 65,300 psi).

API 5L is similar to ASTM A252 in that they are both hydrostatically tested; however, API 5L material is of a much higher quality as it is expected to withstand higher pressures and much higher temperatures than A252 pipe.

The most common grade of API 5L pipe available for structural applications or pile foundations is API Grade 2B or API Spec 2B is an economical alternative of API 5L for pile foundations.

List of API 5L steel material and their yield strength are given below:

Table 3.4: Mechanical Properties of API 5L Pipe Pile Material

Steel grade	**Yield strength N/mm^2**
API X52	358
API X56	386
API X60	413
API X65	448
API X70	483

b) ASTM A252

ASTM Specification A252 describes nominal (average) wall steel pipe piles of cylindrical shape and applies to pipe piles in which the steel cylinder acts as a permanent load-carrying member. For pipe pile according to ASTM A252, three grades of steel, Gr 1, Gr 2, and Gr 3 are used as provided below.

Table 3.5: Mechanical Properties of Pipe Pile (ASTM A252)

	Grade 1	**Grade 2**	**Grade 3**
Tensile Strength, min, psi	50,000	60,000	66,000
Yield Strength	30,000	35,000	45,000

c) ASTM A500

If a large diameter pipe piles are required, but the stringent chemical and testing requirements of the API 5L specification are not required, it may be judicious to use the section as “ASTM A500 Grade C. This allows a mill that has not gone through the rigorous certifications necessary to obtain an API license to produce the material needed for a construction application, which are cost effective. The tolerances in A500 are more stringent than A252 for wall thickness, and also A252 has no tolerance for straightness.

3.8.2 High strength steel for pile foundations

The effective life of a pile may be increased by retaining the same section but supplying it in steel with higher yield strength. The structural design may also exploit this higher strength. All grades of carbon steel have similar corrosion rates. However, the provision, for example, of grade S355GP sheet piles where the calculated stresses require grade S270GP (when the pile is new), will allow an additional 30% loss of thickness to be sustained without detriment. An even greater performance increase can be achieved by specifying S390GP, S430GP or S460GP steels to EN 10248.

However, care is required when considering using a higher grade steel and a corresponding smaller thickness. A reduction in the second moment of area for the thinner section will result in a greater deflection of the pile, which may not then be acceptable. Higher strength steels may require protection from corrosion so sacrificial steel is not advisable for structural strength assessment as per EN 1993-5.

3.8.3 Weldability of Steel

An unlimited suitability of steels for welding cannot be presumed, since the behaviour of steel during and after welding depends not only on the material but also on the dimensions and the shape, as well as on the manufacturing and service requirements of the structural member.

The suitability for arc-welding can be presumed for all sheet piling steel grades, if the general welding standards are observed.

The requirements for the chemical composition of the commonly specified structural steels and, in particular, the limiting values of carbon equivalents, should be selected to facilitate the weldability.

ASTM A53, A500, A1085, and API 5L Grade B, X42, and X52 are all listed as approved base metals for prequalified welding procedure specifications.

According to EN 10025, the carbon equivalent CEV should not exceed the values of the steel grade S 355 as per in the interests of weldability

As the amount of welding work on connected sheet piling is comparatively extensive, the sheet piles are to be manufactured of steel grades with full suitability for fusion welding.

3.9 Other Equivalent steel grades for sheet pilings

Various standard across the Globe has specified the minimum yield value for the particular thickness to be used as the nominal (characteristic) yield strength f_y. Also designers should note that yield strength reduces with increasing plate or section thickness (thinner material is worked more than thick material and working increases the strength. Following tables, provides the (characteristic) yield strength f_y for both hot rolled and cold formed sheet piles, Tie rods and interlocking steel pipe piles.

Table : 3.6 - Design parameters of British/European (BS EN) structural steels

Grade	***fy* (N/mm²), for thickness (mm) less than or equal to**					
	16	**40**	**63**	**80**	**100**	**150**
S235	235	225	215	215	215	195
S275	275	265	255	245	235	225
S355	355	345	335	325	315	295
S420	420	400	390	370	360	340
S460	460	440	430	410	400	380
S500	500	500	480	480	480	440
S550	550	550	530	530	530	490
S620	620	620	580	580	580	560
S690	690	690	650	650	650	630

Table : 3.7 - Design parameters of American (ASTM) structural steels

Grade Ksi [MPa]	***fy* (N/mm²), for thickness (mm) less than or equal to**				
	32	**50**	**65**	**80**	**100**
36 [250]	250	240	230	220	210
42 [290]	290	280	270	260	250
50 [345]	345	335	325	315	305
55 [380]	380	370	360	350	340
60 [415]	415	405	395	385	375
65 [450]	450	440	430	420	410
70 [485]	485	475	465	455	445
100[690]	690	680	670	660	650

Table : 3.8 - Design parameters of API Line Pipes

Grade Ksi [MPa]	***fy* (N/mm²), for thickness (mm) less than or equal to**				
	32	**50**	-	-	-
B [L245]	245	235	-	-	-
X42 [L290]	290	280	-	-	-
X46 [L320]	320	310	-	-	-
X52 [L360]	360	350	-	-	-
X56 [L390]	390	380	-	-	-
X60 [L415]	415	405	-	-	-
X65 [L450]	450	440	-	-	-

Table : 3.9 - Design parameters of Japanese (JIS) structural steels

Grade	***fy* (N/mm²), for thickness (mm) less than or equal to**					
	16	**40**	**75**	**100**	**160**	**200**
400	245	235	215	215	205	195
490	325	315	295	295	285	275
490Y	365	355	335	325	-	-
520	365	355	335	325	-	-

570	460	450	430	420	-	-

Table : 3.10 - Design parameters of Australian/New Zealand (AS/NZS) structural steels						
Grade	***fy* (N/mm²), for thickness (mm) less than or equal to**					
	12	**20**	**32**	**50**	**80**	**150**
250	250	250	250	250	240	230
300	300	300	280	280	270	260
350	350	350	340	340	340	330
400	400	400	360	360	360	-
450	450	450	420	400	-	-
CA220	210	-	-	-	-	-
CA260	250	-	-	-	-	-
CA350	350	-	-	-	-	-
PT430	300	280	280	270	270	250
PT460	305	295	295	275	275	265
PT490	360	340	340	330	330	320
PT540	450	450	420	400	-	-

Table : 3.11 - Design parameters of Chinese (GB) structural steels					
Grade	***fy* (N/mm²), for thickness (mm) less than or equal to**				
	16	**35**	**50**	**100**	**150**
Q235	235	225	215	215	195
Q275	275	265	255	245	225
Q295	295	275	255	235	-
Q345	345	325	295	275	-
Q355	355	345	335	325	-
Q390	390	370	350	330	-
Q420	420	400	380	360	-
Q460	460	440	420	400	-

Chapter 4

Geotechnical Studies for the Site

Soils are made by nature and not by man, and the products of nature are always complex… As soon as we pass from steel and concrete to earth, the omnipotence of theory ceases to exist. Natural soil is never uniform. Its properties change from point to point while our knowledge of its properties is limited to those few spots at which the samples have been collected. In soil mechanics the accuracy of computed results never exceeds that of a crude estimate, and the principal function of theory consists in teaching us what and how to observe in the field.

- Carl Terzaghi

4.1 Geotechnical Investigations

Geotechnical investigation is a pre-requisite for safe and economical design of foundations and ground supported structures. For sheet pile wall, design engineers needs to identify the physical qualities of site in order to determine if site is suitable and safe for the purpose and identify the potential of complexity during execution. As embedded retaining wall derives major loads and supports from the surrounding soil, a structured site investigation along the wall alignment is essential part of the project. The investigation shall be carried out involving the following features;

a) The investigation should accurately reflect the;

- ➢ The nature and sequence of strata;
- ➢ The ground water conditions at the site;
- ➢ The physical properties of underlying strata such as colour, texture and composition of soil;
- ➢ The mechanical properties of strata, such as strength and compressibility, and

- Other specific information, such as the chemical composition of the groundwater, and the characteristics of foundations adjacent to the wall etc.

b) The depth of investigation should comprehend the entire zone of soil or rock affected by the construction.

c) Investigation should adequately explore, if the site is suitable and safe for the construction of wall and identify the potential of complexity which would impede the pile driving during execution.

d) The investigation should be a cooperative effort of the structural and geotechnical engineer familiar with the area, as the behaviour of retaining structure is grossly influenced by soil structure interaction.

In addition to above the main objective of investigation is to produce all relevant geotechnical information and data required for various stages of the project. The information shall be adequate to deal with all identified and anticipated risks. The Information and data shall also help the designer to consider various structural options that are safe and economical.

The magnitude of geotechnical investigation is decided by:

a) Information of geotechnical conditions of the area
b) Size and importance of the project
c) Loads and limiting settlements of various units
d) Prevailing sub-soil conditions

The investigation should be conducted in phases such as feasibility phase and final design phase and the information obtained in each phase shall be advantageously used in planning the later phase as described below. The results of each phase shall be used to "characterize" the site deposits for analysis and design by developing idealized material profiles.

4.2 Stages of Site Investigation

The approach adopted for a particular site investigation, its extent and the techniques used will all depend upon the site-specific circumstances, and the experience and judgment of the design engineer or project manager. There is no single way to carry out an investigation, and inevitably different advisors will adopt different approaches for any particular project.

However, it is usual for the site investigation to be a phased exercise in association with geotechnical and structural engineer. For any project soil investigation is usually performed in several ways, these are termed as:

I. A desk study and site walk-over survey;
II. Preliminary investigation;
III. Detailed investigation;
IV. Supplementary investigation during construction (if required).

4.2.1 Desk Study and Site Walkover Survey

A key component of ground investigation is the desk study and walk-over survey, where all the available existing information on a site is acquired and interpreted, including topographical and geological maps, records of previous site investigations, and current and previous land use (information of constructions, buried electrical cables, trenches, foundations, land fill along with possibility of contaminated land). The walkover survey should also identify critical areas for sinking of additional boreholes such as marshy land, intertidal zone, frequently flooded area etc.

4.2.2 Preliminary Investigation

It is a feasibility phase exploration. In this phase a moderate number of borings should be obtained. Borings should extend deep enough to sample any materials which may affect the performance pile or wall. The exploration program must be sufficient not only to evaluate the design of wall but also assess the stability of the overall system. The data obtained from preliminary investigations should be used for broad qualitative analysis (as discussed in section 4.6 & 4.8), preliminary site characterisation, identification of problematic or suspicious sub-soil strata (described in section 4.7) and basic sub-soil parameter evaluations. For walls where under-seepage is of concern, a sufficient number of the borings should extend deep enough to establish the thickness of any pervious strata. The spacing of borings depends on the geology of the area and may vary from site to site.

4.2.3 Detailed Site Investigation

It is preconstruction engineering and design phase. During this phase, explorations are conducted to develop detailed material profiles and quantification of material parameters. The number of borings should typically be two to five times the number of preliminary borings. The spacing of borings and layout should be controlled by the geologic conditions of site and the characteristics of the proposed structure. Based on the preliminary site characterization and qualitative analysis of site borings should be planned to confirm the location of significant changes in subsurface conditions as well as to confirm the continuity of apparently consistent subsurface conditions. At this time, undisturbed samples should be obtained for laboratory testing and/or in situ tests should be performed. After this stage, final site characterisation with idealised ground profile should be prepared. At this stage, comprehending all the geotechnical with empirical correlations (if required) a simplified and idealised profile shall be prepared to carry out the appropriate analysis and design. ***To analyse is first to idealise***. The idealised ground profile prepared along the proposed wall or structure alignment should include all the necessary parameters, details required for stability analysis and design

4.2.4 Supplementary Investigation and Construction Control

It should be noted that for geotechnical exploration, few boreholes in a site cannot provide fully accurate picture of underlying deposits, obstructions etc. however, sheet pile wall is a continuous structure and presence of localised extremely hard deposits or obstructions may hinder the pile driving and installation process as per design. Therefore, if required supplementary investigation is also made to give a full picture of the site sub-soil formation.

For installation of sheet pile walls, geophysical methods such as seismic method often provide an ability to rapidly define general conditions at modest cost. After finalisation of sheet pile wall alignment, it is imperative to conduct the geophysical investigation to know the underlying obstructions etc. to avoid unnecessary delay during execution.
In some cases, additional exploration may be useful to resolve questions arising during detailed design and also to modify the construction plan, if project demands.

4.3 Depth of Investigation

The site investigation should be carried to such a depth that the entire zone of soil or rock affected by the changes or construction should be adequately explored.

- Investigate to a depth below bottom of wall between 1 and 2 times the wall height or a minimum of 3 m into bedrock. Investigation depth should be great enough to fully penetrate soft highly compressible soils (e.g. peat, organic silt, soft fine-grained soils) into competent material of suitable bearing capacity (e.g., stiff to hard cohesive soil, compact dense cohesionless soil, or bedrock).
- In case of heavy loaded piles, depth of investigation should extend below the anticipated pile or shaft tip elevation a minimum of 6 m. For shafts supported on or extending into rock, a minimum of 3 m of rock core, or a length of rock core equal to at least five times the shaft diameter for isolated shafts or two times the maximum shaft group dimension, whichever is greater, shall be extended below the anticipated shaft tip elevation to determine the physical characteristics of rock within the zone of foundation influence.
- Minimum 15.0m depth shall be investigated in the case of liquefaction study.

4.4 Horizontal spread of geotechnical exploration

Horizontal spread of investigation points depends on the variability in subsoil conditions and sensitivity of structures towards differential settlements. The following may be used as a guide to decide the investigation points.

a) The borehole / other test locations may be spaced at 20m to 30m in the case of residual soil deposits where lateral variations are abrupt and significant

b) The spacing may be 60m to 100m in the case of natural alluvial deposits. Lesser spacing may be warranted in the area very close to river courses

c) 30 to 50m spacing in the case of marine deposits

d) 20 to 30m spacing for structures sensitive to differential settlements

In totality, geotechnical investigation is a combination of field investigation (insitu) and laboratory tests and they are complementary to each other. For insitu test Standard Penetration Test, Vane Shear Test, Permeability test etc. are very common techniques.

4.5 Field Investigation

4.5.1 Standard Penetration Test (SPT)

The Standard Penetration Test, known as the SPT, is commonly for subsurface investigations and geotechnical designs. It is one of the most broadly used tests world-wide to characterize in-situ soil strength. While other in-situ tests are available, CPT, CPTU and dilatometer to mention a few, only the SPT test enables the drill crew to retrieve soil samples. The SPT test is made by dropping a free-falling hammer weighing 65 Kg onto the drill rods from a height of 750mm to achieve the penetration of a standard sample tube 300 mm into the soil. The sampler is driven using the hammer for 450 mm recording the number of blows for every 150 mm. The number of blows for the last 300 mm drive shall be reported as N value.

The advantage of the SPT tests is that'

- The drillers can collect samples for further classification and laboratory testing.
- This is simple and economical test
- SPT N values are empirically correlated with geotechnical design parameters such as soil density, consistency, friction angles, undrained shear strength, Young's modulus, shear modulus, settlement of shallow and deep foundations in sand, bearing capacity values, and to provide an index of soil liquefaction resistance of cohesionless soils.
- It can also be used to determine the unconfined compressive strength of cohesive soils.

This monograph, however, does not provide detailed procedures for field investigations or SPT method. For procedure and details of field investigations, standard geotechnical text books and relevant Codes of Practices should be referred.

4.5.2 Laboratory Testing Program

Prior to planning the laboratory-testing program, it is again necessary to review the design needs of the project. Specifying unnecessary laboratory tests will add time and cost to the project and consume samples. The laboratory test may include the results of moisture content, Atterberg limits, grain size distribution, unit weight, void ratio, shear strength and compressibility.

Once a list of necessary tests has been developed and the field program has been executed, the engineer should review the results to identify "critical areas". Critical areas correspond to borings/locations where the results of the laboratory tests could result in a significant change in the proposed design. Samples from these critical areas should be identified for performance testing. For example, if unexpected clay deposits were discovered in the area of a proposed bulkhead, consolidation testing would be required to assess potential settlements. If the test results show that settlements may be excessive along the proposed alignment, location of wall has to be moved, or redesigned, or deep foundation support should be considered.

It is necessary to select samples for laboratory analysis that will accurately characterize the site. In heterogeneous areas, many samples may be required to obtain comprehensive parameters; in homogeneous areas, few samples may be required. Laboratory-testing program should be performed on representative and critical specimens from the site.

For generation of detailed subsurface cross sections, in-situ testing results, and laboratory index test results are very useful tools.

4.6 Challenges in Geotechnical Investigation

The nature of soil is very complex, infact it is a two or even three phase material. It is much more complex than the classical structural materials such as steel, concrete and even timber. Therefore, obtaining more reasonable and accurate geotechnical information is a challenging task for geotechnical engineers. The most common challenges are;

a) **Maintaining the Quality of Field Investigation** - Field investigation is primarily dependent on the experience and ability of the drilling personnel, supervising geotechnical engineer, and adequacy of the plant and tools being used.

b) **Obtaining Undisturbed Samples** - In cohesive soils, the effects of unavoidable sample disturbance in even so-called “high quality” undisturbed samples are sometimes difficult to assess. In cohesionless soils, undisturbed sampling is still an unsolved problem in everyday practice.

c) **Quality of Laboratory Test -** For some laboratory tests, operator error could have a significant influence on the end results.

4.7 Application of Empirical Correlations to Determine the Geotechnical Parameters

Empirical correlations are widely used in geotechnical engineering practice as a tool to estimate the engineering properties of soils. Useful correlations exist between the index properties obtained from simple routine testing and the strength and deformations properties of cohesive soils among others.

The empirical correlations can also be used to compare against the values determined from laboratory and in situ tests. There are so many empirical equations and graphs available, which are regularly being used in the designs worldwide. These are derived based on laboratory or field data, past experience, and good judgement.

The US Army, Navy has done excellent engineering work, and invest significantly in research and development. All their design guides, empirical equations, and charts are very well proven and tested. Also, the Canadian Foundation Engineering Manual is a well reputed design manual. They (e.g. NAVFAC DM-7.1, DM-7.2, DM-7.3) are valuable additions to the professional practice.

In day to day practice, the general applications of empirical for qualitative evaluation of soil parameters are as follows;

- Physical and mechanical properties of cohesive (fine-grained) soils greatly depend on the water content. The Atterberg limits and consistency indices are used for classifications of cohesive soils. They provide information for interpreting several soil mechanical and physical properties such as shear strength, compressibility, shrinkage and swelling potentials of soil.

  The plasticity index is commonly used in strength correlations; the liquid limit is also used primarily for consolidation estimates. In practice, the overall effects of these factors are indirectly expressed with the index properties. A high *LL* or *PL* in a soil is indicative of a more "clay-like" or "plastic" behaviour.

  The liquid and plastic limit values, together with W_M (natural moisture content), are useful in predicting whether a cohesive soil mass is pre-consolidated. From this we might deduce the following:

If W_M is close to W_L,	***soil is normally consolidated***
If W_M is close to Wp,	***soil is some- to heavily over-consolidated***
If W_M is intermediate,	***soil is somewhat over-consolidated***

If W_M is greater than W_L soil is on verge of being a viscous liquid

- Generally, the frictional resistance to shearing, as expressed by ϕ', can be expected to decrease as the content of platy clay minerals increases in the soil mass. With increasing content of platy clay particles both the liquid limit $w_L$and the plasticity index I_p will increase, and hence a correlation between ϕ' and $w_L$or I_p can be expected. Simple correlations between the plasticity index and the drained peak strength parameters in terms of $\emptyset'_{oc}$ and c'_{oc} have been proposed on the basis of a comprehensive database of triaxial compression tests on undisturbed over-consolidated clays of very low to extremely high plasticity.
- From simple index properties (LL, PL, PI and void ratio), one can get a fair idea about the shear strength and consolidation characteristics and stress history of clay at little or no cost.

Above correlations gives a qualitative indication of sub-soil parameters and other methods must be used if a quantitative value is required.

4.8 Assessment of Difficult Soils

Certain soils are identified as problematic soils wherein, the presence of such soil poses geo-hazards or geo-risk. These soils shall be identified and investigated further for their performance. These soils are

a) Expansive soil
b) Peaty soil
c) Liquefiable soil
d) Collapsible soil
e) Sensitive soil

4.8.1 Expansive Soil Properties

Soils with the following properties are likely to be expansive type.

a) plasticity index PI more than 15
b) 30% or more particles smaller than 75 microns
c) 10% of the particles smaller than 5 microns in size, and

d) the differential free swell index more than 20%

Swell pressure tests, activity index, mineralogical studies, ground water fluctuations, and the active depth are needed for better understanding of expansive soil. Peaty soil is identified by its dark colour, large organic content, spongy nature, and very high moisture content. These have low strength and high compressibility.

4.8.2 General Properties of Liquefiable Soil

Cohesionless soil deposits are susceptible to liquefaction if they are loose enough to be contractive and are of sufficiently low permeability to experience no significant drainage during the period of ground shaking/ seismicity.

Based on preliminary geotechnical investigation at site, first qualitative analysis of sub-soil should be carried out to determine the liquefaction potential of ground, which may be based on guidelines outlined in "*Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analysing and Mitigating Liquefaction Hazards in California*" and NCEER/ NCF.

If qualitative analysis indicates the hazard potential than detailed investigation should also be planned to collect the information for further quantitative analysis. Following screening criteria are generally applied for qualitative analysis against liquefaction hazard.

- Liquefaction is possible for clean sands having relative density lesser than 40% to 60 %; the susceptibility within the range depends on confining pressure.
- Clean fine sands and non-plastic silty sands containing less than 5% fines are the most susceptible to liquefaction, because they are likely to be deposited in a loose condition and to have a permeability low enough to permit little drainage during the period of ground shaking.

These types of sub-soil strata should to be further investigated for liquefaction potential and lateral spreading. Ground water table studies are necessary to identify the thickness of liquefiable soil.

- Coarse sand and gravels have lesser chance of liquefaction than the fine sand as they have much higher permeability.
- If clayey materials are encountered during site exploration, those materials may be considered as non-liquefiable.
- Also, clayey soils having any of the following characteristic are not susceptible to severe strength loss due to liquefaction:
    a) Percentage of fine contents (finer than 0.005 mm) more than 15 percent.
    b) Liquid Limit greater than 35
    c) Water Content greater than 0.9 x Liquid Limit.

4.8.3 Collapsible Soil

Loose arid and semi-arid deposits formed by the wind are likely to have poor density, but stable under dry conditions. This soil becomes unstable when wet or saturated. Lateritic soils with highly vesicular structure are likely to be significantly collapsible. Flash floods and mudflows result in collapsible soil deposits from alluvial soils with more silt content. Such deposits may be investigated for compressibility and degree of collapse. Single or double oedometer tests may be one of the procedures.

4.8.4 Sensitive Soil

Sensitive soils are high plasticity clays with high water content. They have very low remoulded strength. This type of soil is also susceptible to liquefaction and lateral spread under seismic conditions. The sensitive soil is identified from remoulded strength determination and other index properties.

4.8.5 Some Important Points for Assessment of Geotechnical Parameters.

Classification and index tests (water content, Atterberg limits, and grain size) should be performed on most or all samples and shear tests should be performed on selected representative undisturbed samples. Where settlement of fine-grain foundation materials is of concern, consolidation tests should also be performed. The strength parameters ϕ and c is not intrinsic material properties but rather is parameters that depend on the applied stresses, the degree of consolidation, and the drainage conditions during shear. Consequently, their values shall be based on laboratory tests that appropriately model these conditions as expected in the field.

4.8.6 Properties of Coarse-grained Materials

Coarse grain materials such as sands, gravels, and non-plastic silts are sufficiently pervious that excess pore pressures do not develop when stress conditions are changed. Their shear strength is characterized by the angle of internal friction (ϕ). For mix type of soil, e.g., (c- ϕ) soil, shear strength are determined from consolidated, drained (CD) tests. Failure envelopes plotted in terms of total or effective stresses are the same, and typically exhibit a zero *'c'* value and a ϕ value in the range of 20 to 45 degrees.

The value of ϕ for coarse grain soils varies depending predominately on the particle shape, gradation, and relative density. Because of the difficulty of obtaining undisturbed samples of coarse-grained soils, the ϕ value is usually inferred from in situ tests or conservatively assumed based on material type. The details for estimation of shear strength based on approximate relationships between the relative density, standard penetration resistance (SPT), angle of internal friction, and unit weight of granular soils are given in Table 4.1 (after Mitchell and Katti).

Table 4-1
Granular Soil Properties (after Mitchell and Katti)

Compactness	**Relative Density (%)**	**SPT `N' (blows per 30 cm)**	**Angle of Internal Friction (deg.)**	**Dry Unit Weight (KN/m^3)**
Very Loose	0 - 15	0 - 4	< 28	< 14
Loose	16 - 35	5 - 10	28 - 30	14 - 16
Medium	36 - 65	11 - 30	31 - 36	16 - 18
Dense	66 - 85	31 - 50	37 - 41	18 - 20
Very Dense	86 - 100	> 51	> 41	> 20

Figure 4.1 shows correlation between ϕ and relative density, and unit weight for various types of coarse-grain soils.

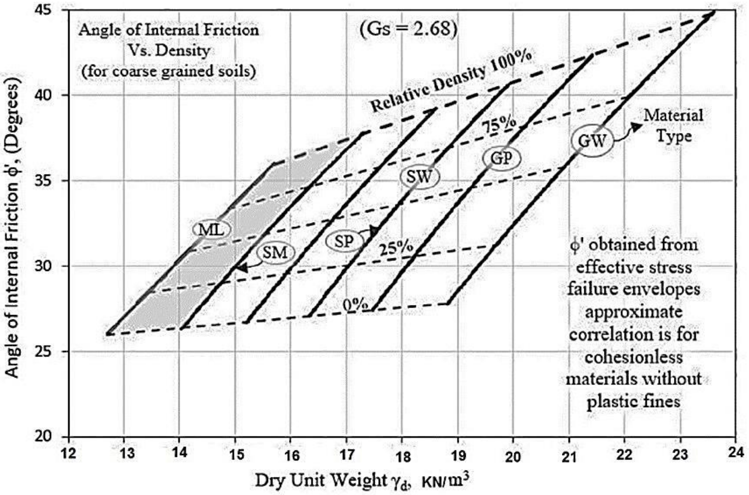
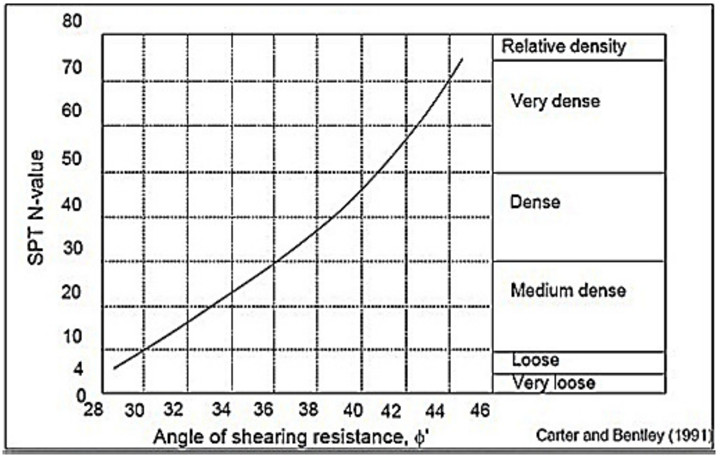


Figure 4.1: Properties of Cohesionless Soil (after U.S. Navy 1971)

Figure 4-2 shows another correlation between corrected or normalised value of 'N' and the ϕ, for medium grained clean sand. Material finer or coarser may have different values and computed ϕ value of soil shall be modified. The ϕ, value shall be reduced up to 5deg. for clayey sand and increase by 5 deg. for gravelly sand.



Carter and Bentley (1991)

Figure 4.2: Relationship between ϕ and N

For identification of soil type like ‘clean sand’, ‘silty sand’ the constituent of soil should be as following;

Clean Sand	If fraction of fines is negligible or in very small proportion (< 5%)
Silty or clayey Sand	If fraction of fines (silt or sand) in appreciable quantity (>12 %)
Sand with Silt or clay	If fraction of fines lies between 5 to 12%, it is identified as ‘Sand with Silt’ or ‘Sand with clay’

In the absence of fully undisturbed soil samples, where site-specific correlations are desired for important structures, laboratory tests may be performed on samples re-compacted to simulate field density.

Also, the wall friction angle, δ, is usually expressed as a fraction of the angle of internal friction, ϕ.

Table 4-2 shows the smallest ratios between δ and ϕ determined in an extensive series of tests by Potyondy (1961). It shows angle of wall friction for various soils against steel and concrete sheet pile walls.

Table 4-2: Values of δ for Various Interfaces
(after US department of the Navy 1982)

Soil Type	**δ (deg.)**
Clean gravel, gravel sand mixtures, Well-graded rock-fill with spalls	22
Clean sand, Silty sand-gravel mixture, single size hard rock-fill	17
Silty sand, gravel or sand mixed with silt or clay	14
Fine sandy silt, non-plastic silt	11

From laboratory test results, the particle size distribution according to the shape of the distribution curve has great significance in determination of permeability of soil. The effective particle size of a soil is governed by the size of the smallest 10% fraction of the soil. This size (D_{10}) is related to permeability and thus to permeability of the soil.

- **Soil permeability** can be assessed from the particle size distribution curve. This is done by taking the square of the 10% fraction particle size (D_{10}) and assuming this to be the rate of flow. For instance, for a D_{10} particle size of 0.2mm, permeability would be $(0.2)^2 = 0.04$ or k=4 x 10^{-2} cm/sec.

4.8.7 Properties Fine-grain materials (cohesive soils)

Localized tests such as torvane (TV), pocket penetrometer (PP) and miniature vane (MV) in soil samples provide a general quick measurement to the undrained shear strength values in most cohesive soils. UU triaxial tests are generally performed on larger cohesive soil mass and the mass is compressed to failure and provide a more accurate guide to the in-situ undrained shear strength. Higher strength values are obtained by tri-axial tests may be due to sand/silt content, unsaturated sample or a combination of both factors.

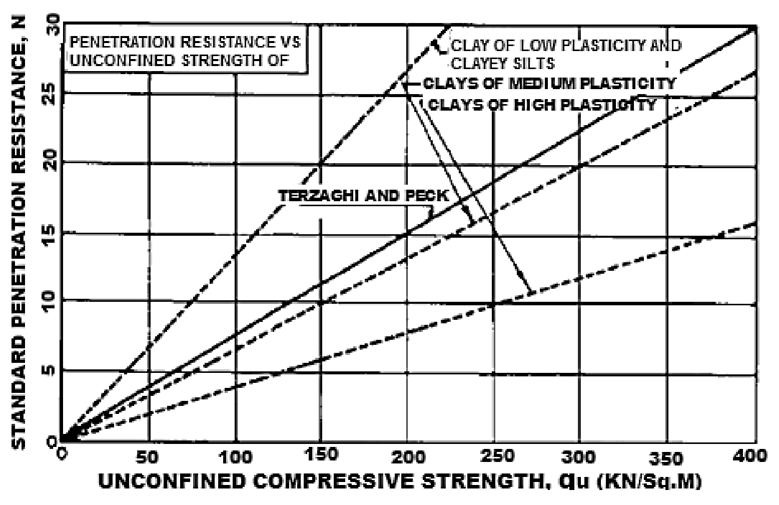


Figure 4.3 Correlation of Unconfined Compressive Strength of Clayey soil and SPT 'N' Blow Count

Sometimes recovery of undisturbed soil samples is very difficult in hard clayey strata, the SPT tests may be correlated with Figure 4-3 (after Design Manual 7.01: NFEC, U.S) to estimate the undrained shear strength, co-relating the SPT blow count with plasticity index of soil.

Table 4-3: Index Properties of Clayey Soil

Clay Type	Liquid Limit (WL %)	Plasticity Index (PI %)
Low Plasticity	< 35%	< 12%
Med. Plasticity	35% - 50%	12% - 23%
High Plasticity	>50%	>23%

The shear strength of fine-grained soil, such as clays and plastic silts, are considerably more complex than coarse-grain soils because of their significantly lower permeability, higher void ratios, and the interaction between the pore water and the soil particles.

4.9 Time-dependent behaviours of cohesive soils

Fine-grain soils subjected to stress changes develop excess pore pressures because their low permeability. Thus, their behaviour is time dependent due to their low permeability, resulting in different behaviour under short-term and long-term loading conditions. For short term loadings, undrained shear strength of soil is applied.

However, during long term loading physio-mechanical behaviour of clay get altered. When over-consolidated hard or stiff clay strata is subjected to stress change and associated shear and/or volumetric plastic strains (or both), causes breaking or destructuring of inter-particle bonds of clay and loss of the extra strength achieved due to over-consolidation. The phenomenon is term as softening and strain-softening of clay.

i. This is time dependent process and the behaviour of soil changes from brittle to ductile and implies a variation in mechanical parameters (such as Modulus of elasticity, cohesion values and water content) of soil. This deterioration is usually thought to be responsible for partial or complete vanishing of the cohesion and final process causes the decrease in the basic friction angle from the critical to the residual value. During design, the residual shear strength of clay shall also be evaluated using graph in Fig 4.4 to assess the critical shear parameters for design.

ii. The second factor, higher void ratio, generally means lower shear strength. Furthermore, in some (sensitive) clays the loose structure of the clay may be disturbed by construction operations leading to a much lower strength and even a liquid state.

iii. Since an undrained condition may be expected to occur under "fast" loading in the field, it represents a "short-term" condition.

iv. *S*u increases with depth (or effective stress) and this is commonly expressed with the ratio "*S*u/σv" (σv denotes the effective vertical stress). This ratio correlates roughly with plasticity index and over-consolidation ratio in Table 4-4.

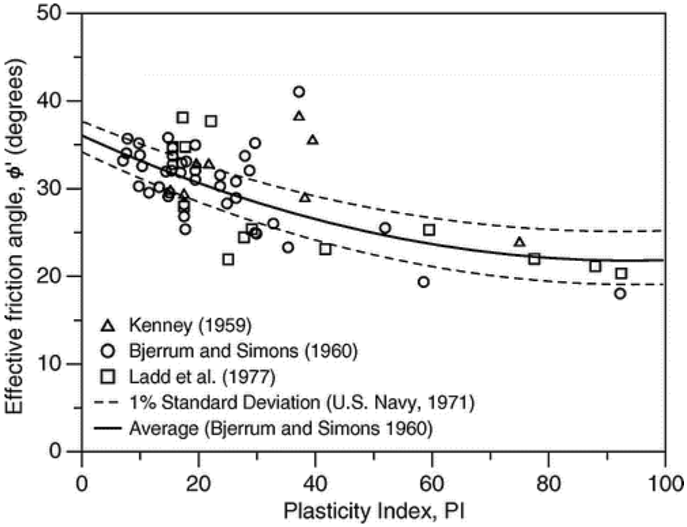


Figure 4.4 Effective angle of friction versus PI for normally consolidated clays

v. The drained shear strength of normally consolidated clays is similar to that of loose sands (*c′ =0*), except that ϕ is generally lower. An empirical correlation of the effective angle of internal friction, ϕ′, with plasticity index for normally consolidated clays is shown in Figure 4.4. The drained shear strength of over-consolidated clays is similar to that of dense sands (again with lower ϕ'), where there is a peak strength (c' nonzero) and a "residual" shear strength ($c' = O$).

vi. Finally it shall be noted that, the general approach in solving problems involving clay is that, unless the choice is obvious, both undrained and drained conditions are analysed separately. The more critical condition governs the design. Total stresses are used in an analysis with undrained shear strength (since pore pressures are "included" in the undrained shear strength) and effective stresses in a drained case; thus such analyses are usually called total and effective stress analyses, respectively.

vii. Certain materials such as “clay shales” exhibit greatly reduced shear strength once shearing has initiated. For walls founded on such materials, sliding analyses should include a check using residual shear strengths.

To assess the consolidation settlement around the wall, it shall be noted that the behaviour of over-consolidated clays are related to the past effective stress and exhibit the shear strength corresponding to a stress level closer to the pre-consolidation pressure rather than the current stress. For a higher stress, above the pre-consolidation pressure, they behave like normally consolidated clays.

Table 4-4 Over-consolidation from Undrained strength ratio (after Ladd et al., 1977)

Over-consolidation ratio	**$Su/\sigma v$**		
	OH Clays	**CH clays**	**CL Clays/ silts**
1	0.25 to 0.35	0.2 to 0.3	0.15 to 0.2
2	0.45 to 0.55	0.4 to 0.5	0.25 to 0.35
4	0.8 to 0.9	0.7 to 0.8	0.4 to 0.6
8	1.2 to 1.5	0.9 to 1.2	0.7 to 1.0
10	1.5 to 1.7	1.3 to 1.5	0.8 to 1.2

Solved Examples

Example 4.1

A test on representative soil sample is carried out to monitor the properties of reclaimed fill. The soil sample is predominantly fine silty sand. The measured volume and mass of soil sample is 25cc and 46gms respectively. After oven drying it for 24 hours at 105ºC, the measured mass is 39.21gms. Relative density of fill material is expected to vary from 35% to 50%. Determine the range of angle of friction of reclaimed earth that may develop at site. Assume specific gravity of soil as 2.7.

Solution:

Total mass of soil sample (M) = 46.00 gms.
Volume of soil sample (V) = 25 cc.
Mass of dry soil sample =39.21 gms.
Moisture content in soil sample (w) = 46.0 - 39.21
= 6.79gms.
Percentage of moisture content = 17.3%
Bulk density of soil sample (ρ) = $\frac{M}{V}$ = $\frac{46.0}{39.21}$
= 1.84 gm/cc
Dry density of soil sample (ρ_d) = $\frac{\rho}{(1+w)}$ = $\frac{1.84}{(1+0.173)}$
= 1.57 gm/cc (15.7 KN/m^3)

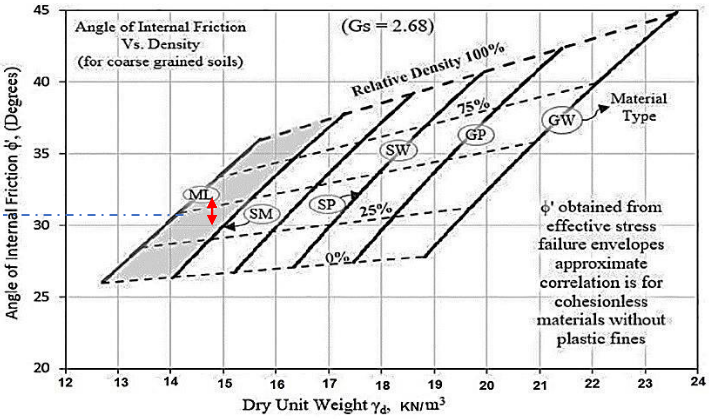


Figure E: 4.1

From figure E: 4.1 as indicated above the expected ϕ value of the fill will be around 30° - 31°.

Example 4.2

In example 1.1, if expected relative density of sand fill is around 60 - 75% and measured dry density of soil is 17 KN/m^3**,** what should be the friction angle of filled-up material.

Solution:
From figure E: 4.1, the expected value of fill will be around 34°.

Example 4.3

In example 1.1, if expected relative density of predominantly non-plastic silt with fine is around 75% and dry density is 15 KN/m^3**,** what should be the expected friction angle of filled-up material.

Solution:
From figure E: 4.1, the expected value of fill will be around 34°.

Example 4.4

Underlying clayey soil strata have corrected SPT 'N' value as 24. The liquid limit and plastic limit of soil sample is 28% and 17%. What shall be the expected shear strength of soil for;

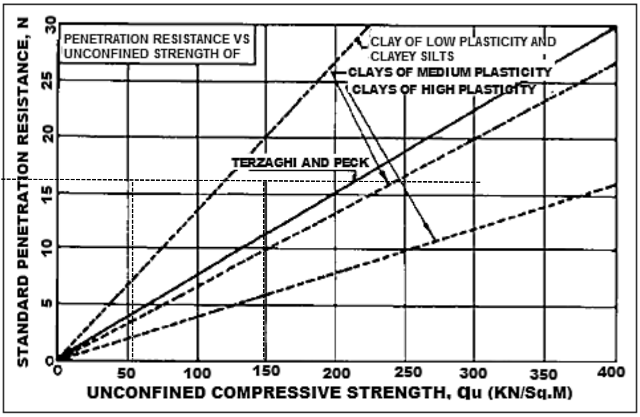
a) Short term / undrained condition
b) Long term / drained condition

Solution:

a) Shear strength for short term / undrained condition

Liquid Limit of soil = 28 %
Plastic Limit of soil = 17 %
Plasticity Index = 28 % - 17 %
= 11 %

Plasticity Index is low and liquid limit is < 30%



From above figure (Fig 4-3) by NFEC, the unconfined compressive strength of soil; qu = 180 KN/m^2

Shear strength of soil = qu/2

= 90 KN/m^2

b) Shear strength for long term / drained condition

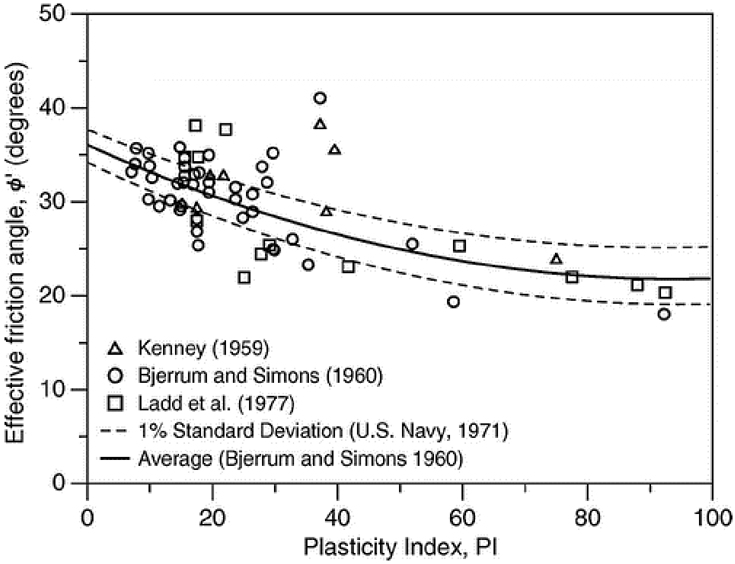


Figure E-4.3

Inferring figure E-4.3 above, the shear parameter for long term and drained condition of soil are as follows;

c = 0 KN/m^2 and φ = 33 deg.

Example 4.5

In example 4.4, if expected liquid limit and plastic limit of soil sample is around 50% and 28% and SPT value is 16. What shall be the expected shear strength of soil for short term and long term condition?

Solution:

a) Shear strength for short term / undrained condition

Liquid Limit of soil = 50 %
Plastic Limit of soil = 28 %
Plasticity Index = 50 % - 28 %
= 22 %

Plasticity Index is 22% and liquid limit is around 50%; therefore this is a medium plastic soil stratum.

From above figure (Fig 4-3) by NFEC, the unconfined compressive strength of soil; qu = 210 KN/m^2
Shear strength of soil = qu/2
= 105 KN/m^2

b) Shear strength for long term / drained condition

Inferring figure E-4.3 above, the shear parameter for long term and drained condition of soil are as follows;

c = 0 KN/m^2 and φ = 31 deg.

Example 4.6

In example 4.4, if liquid limit and plastic limit of soil sample is revised to 73% and 40% respectively, what shall be the revised shear strength of soil?

Solution:

a) Shear strength for short term / undrained condition

Liquid Limit of soil = 78 %
Plastic Limit of soil = 40 %
Plasticity Index = 78 % - 40 %
= 38 %

Plasticity Index is 38% and liquid limit is around 78%; therefore the plasticity of soil is high.

From above figure by NFEC, the unconfined compressive strength of soil will be high, however, without lab test results
the qu value will be limited to = 210 KN/m^2
Shear strength of soil will be = qu/2
= 105 KN/m^2

b) Shear strength for long term / drained condition

Inferring figure E-4.3 above, the shear parameter for long term and drained condition of soil are as follows;

c = 0 KN/m^2 and $\boldsymbol{\varphi}$ = 28 deg.

Chapter 5

System Loads

> *Nature never "fails." Nature complies with its own laws. Nature is the law. When Man lacks understanding of Nature's laws and a Man-contrived structure buckles unexpectedly, it does not fail. It only demonstrates that Man did not understand Nature's laws and behaviours. Nothing failed. Man's knowledge or estimating was inadequate.*
>
> \- R. Buckminster Fuller

The basic philosophy governing the design of a retaining system is that, it should be designed to sustain with a defined probability of all actions likely to occur within its life span. The structure should maintain stability during unprecedented actions fulfilling the durability criteria. Therefore, estimation of the magnitude of loads with reasonable accuracy is very important. The loads governing the design of a sheet pile wall arise primarily from the soil and water surrounding the wall and from other influences such as surface surcharges and external loads applied directly to the piling. All necessary details should be obtained of static, transient and dynamic loads that may be applied externally to the earth retaining structure. Methodologies for evaluating these loads are broadly discussed in this chapter.

5.1 General

Earth pressures reflect the state of stress in the soil mass. Earth pressures for any given soil-structure system may vary from an initial state of stress referred to as at-rest, K_0, to minimum limit state referred to as active, K_a, or to a maximum limit state referred to as passive, K_p.

The coefficient of lateral earth pressure, K, at any point, is defined as the ratio of the horizontal effective stress, σ'_h to the vertical effective stress, σ'_v, at that point.

$$K = \sigma'_h / \sigma'_v \tag{5.1}$$

There are three types of earth pressures on the basis of the movement of the wall.

i) Earth Pressure at rest

ii) Active Earth Pressure

iii) Passive Earth Pressure, as explained below;

5.2 Pressure at-Rest

At-rest pressure refers to a state of stress where there is no lateral movement or strain in the soil mass. In this case, the lateral earth pressures are the pressures that existed in the ground prior to installation of a wall. For normally consolidated soils, the coefficient of earth pressure-at-rest K_0 is given approximately by the equation:

$$K_o = 1 - \sin \emptyset \tag{5.2}$$

This is a case of zero lateral strain during deposition of the soil. K_o is known to increase with over-consolidation of the soil as the stress changes. For over-consolidated soil, the $K_o O_c$ can be expressed as with overconsolidation ratio as:

$$K_o O_c = (1 - \sin \emptyset)\, OCR^m \tag{5.3}$$

Where, *m* is an exponent related to the soil type and can be estimated as approximately equal to $1 - \sin \emptyset$.

The magnitude and use of K_o should be adopted judiciously as the construction of retaining structures cannot occur without some change in the horizontal stress state through excavation, and backfilling.

5.3 Active pressures

Active soil pressure is the minimum possible value of horizontal earth pressure at any depth. This pressure develops when the walls move or rotate away from the soil allowing the soil to expand horizontally in the direction of wall movement. A number of theories have been developed and used to evaluate the lateral earth pressures on rigid retaining structures. The two most common are those developed by Coulomb

(1776) and Rankine (1856). Coulomb's approach to the lateral earth pressure is based on assumption that the resultant earth force acts at an angle parallel to the angle of wall friction, measured normal to the back face of the wall. The assumptions for Rankine's solution are nearly identical except that Rankine did not include friction along the back of the wall and the resultant active force was considered to act at an angle parallel to the backfill slope.

For active earth pressure calculations, both approaches can yield reasonable results, although the Rankine solution will generally be more conservative. It represents the failure condition at which the shear strength in the soil is fully mobilized in resisting gravity forces. The lateral strain (expansion) required to mobilize the soil strength is relatively small, but is nevertheless only possible in structures that are not rigidly restrained from rotation or translation.

The ratio of lateral to vertical effective stress in a granular soil under active failure conditions,K_a, can be obtained from the formulae given below. Wall friction can only mobilize by movement between the wall and the soil. In practice, the effect of wall friction is often ignored and the Rankine solutions are applied. If the effect of wall friction is to be included, the relative movement between the wall and backfill should be considered in detail.

For stratified soils, K_a value can be determined for each soil strata. In general, the lateral earth pressure at any depth is equal to $K_a \sigma_z$ where σ_z is the vertical effective stress at depth z.

Coulomb solution for active earth pressure coefficient:

$$\mathrm{K}a = \frac{\sin^2(\alpha+\emptyset)}{\sin^2\alpha.\sin\,(\alpha-\delta)\left[1+\sqrt{\frac{\sin(\emptyset+\delta).\sin\,(\emptyset-\beta)}{\sin(\alpha-\delta).\sin\,(\alpha+\beta)}}\right]^2} \tag{5.4}$$

$$Pa = \frac{1}{2}.\gamma H^2.Ka \tag{5.5}$$

Rankine solution for active earth pressure coefficient:

$$Ka = \cos\beta \,.\, \frac{\cos\beta - \sqrt{\cos^2\beta - \cos^2\emptyset}}{\cos\beta + \sqrt{\cos^2\beta - \cos^2\emptyset}} \qquad (5.6)$$

Note that if δ = 0, β = 0 and α = 90° then both Coulomb and Rankine solutions produce:

$$Ka = \frac{1-\sin\emptyset}{1+\sin\emptyset} = \tan^2(45 - \emptyset/2) \qquad (5.7)$$

In cohesive soil with $\phi = 0$, $\delta = 0$, $\alpha = 90^\circ$, and $\beta = 0$, the total active earth pressure, pa is equal to the total vertical stress minus twice the undrained shear strength, Su as follows:

$$P_a = \sigma_z - 2\mathrm{Su} \qquad (5.8)$$

In a soil exhibiting both cohesion and friction with $\delta = 0$, $\alpha = 90^\circ$, and $\beta = 0$, the active effective earth pressure is as follows:

$$P_a = K_a\sigma_z - 2c\sqrt{K_a} \qquad (5.9)$$

These expressions, however, indicate that the pressures near the top of the wall can be negative. In practice, such calculated negative pressures are disregarded for cohesive soils and supposed to form tension cracks in such situations. Figure 5.1 illustrates theoretical and possible pressure distributions for design.

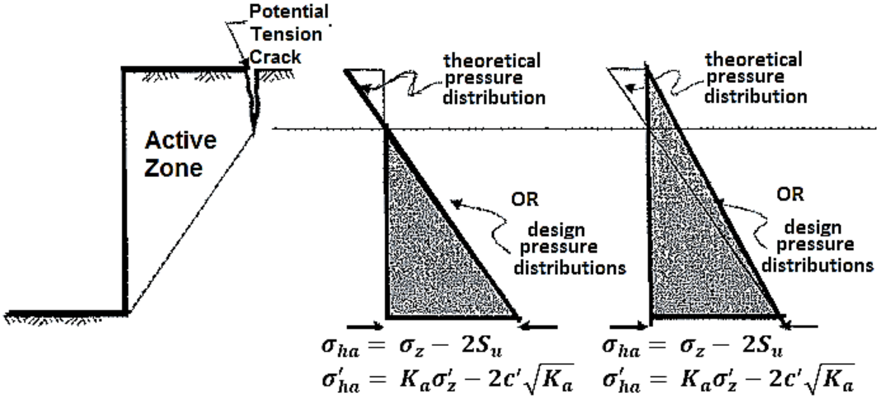

Potential Tension Crack
Active Zone
theoretical pressure distribution
OR
design pressure distributions
theoretical pressure distribution
OR
design pressure distributions
$\sigma_{ha} = \sigma_z - 2S_u$
$\sigma'_{ha} = K_a\sigma'_z - 2c'\sqrt{K_a}$
$\sigma_{ha} = \sigma_z - 2S_u$
$\sigma'_{ha} = K_a\sigma'_z - 2c'\sqrt{K_a}$

Figure 5.1 Calculated and assumed active earth pressure distributions for cohesive and cohesive-frictional soils.

5.4 Passive pressures

Passive (soil) pressure is the maximum possible horizontal pressure that can be developed at any depth from a wall moving or rotating toward the soil and tending to compress the soil horizontally. The ratio of lateral to vertical effective stress in a granular soil under passive failure conditions, K_p can be obtained from the formulae given in Figure 5.10 for retaining structures. In practice, the effect of wall friction is often ignored and the Rankine solutions are applied, especially related to passive earth pressure calculations. During passive failure, the failure surface is not the planar geometry assumed by both Coulomb and Rankine theories.

Coulomb solution for passive earth pressure coefficient:

$$\mathrm{K_p} = \cos(\delta - \mathrm{i}) \frac{\sin^2(\alpha - \emptyset)}{\sin^2\alpha . \sin(\alpha + \delta)\left[1 - \sqrt{\frac{\sin(\delta+\emptyset).\sin(\emptyset+\beta)}{\sin(\alpha+\delta).\sin(\alpha+\beta)}}\right]^2} \tag{5.10}$$

$$P_p = \frac{1}{2}.\gamma H^2.K_p \tag{5.11}$$

Rankine solution for passive earth pressure coefficient:

$$Kp = \cos\beta \frac{cos\beta + \sqrt{cos^2\beta - cos^2\emptyset}}{cos\beta - \sqrt{cos^2\beta - cos^2\emptyset}} \tag{5.12}$$

Note: $\boldsymbol{K_p}$ in *horizontal* direction

Note that if $\delta = 0$, $\beta = 0$ and $\alpha = 90°$ then both Coulomb and Rankine solutions produce:

$$Kp = \frac{1+\sin\emptyset}{1-\sin\emptyset} = \tan^2(45 + \emptyset/2) \tag{5.13}$$

In cohesive soil with $\phi = 0$, $\delta = 0$, $\alpha = 90°$, and $\beta = 0$, the total passive earth pressure, P_p is equal to the total vertical stress plus twice the undrained shear strength, Su as follows:

$$P_p = \sigma.z + 2S_u \tag{5.14}$$

In a soil exhibiting both cohesion and friction with $\delta = 0$, $\alpha = 90°$, and $\beta = 0$, the passive earth pressure is as follows:

$$P_p = K_p \sigma_z - 2c\sqrt{K_p} \tag{5.15}$$

5.4.1 Conditions affecting the lateral earth pressures

The lateral earth pressure mobilised on a retaining structure are precisely related to;

i) Condition of soil / backfill material for retaining wall
ii) Wall movement relative to the soil
iii) Shear strength developed between the wall and the soil
iv) Shear strength of the soil itself
v) Wall, soil, groundwater condition
vi) External loads

The extent of the earth pressure exerted on the wall depends, on the above listed physical factors and strength properties of the soil and other factors are described below;

5.5 Condition of Backfill for Retaining Wall

The term "backfill" is a general term applying to all soil that is near enough to a newly constructed retaining structure to affect its design and performance. It includes material that remains in place as well as material brought to the site for placement during construction. It includes material in front of the wall that will provide resistance as well as material behind the wall that supports surcharge loads and itself generates active loads against the wall.

The ideal wall would contain an idealized soil lightweight, free draining, well graded, homogeneous with optimum engineering properties. This combination is rarely found insitu and often prohibitively expensive to create. The advantages of certain soil types for backfill are well established and these materials should be utilized if at all possible.

Terzaghi listed five general classes of backfill that might be encountered in retaining wall work:

5.5.1 Coarse-grained soils without fines

Very permeable and typically clean sand and gravel are classified in this category. This is the most desirable backfill, either as an insitu material or as an import into the site. The engineering properties of this soil are well established or can be readily determined. They drain freely and remain stable over the long term. The amount of laboratory testing required for design is a minimum for these soils. Cohesionless Soils classified as GW, GP, SW, or SP, Unified Soil Classification System, are excellent backfill materials. When using such soils, theoretical earth pressures are valid for design purposes. If required, these backfill materials can be moderately compacted using deep compaction technique.

5.5.2 Sandy Clays and Clayey Sand

The soil type is same as above, but of low permeability due to presence of silt. This is more typical of fill which might be obtained from dredging operations or which might exist in many marine locations. While not having the desirable free-draining characteristics, this fill is often all that is available at reasonable cost. It is best used in conjunction with selected material placed against the wall for better drainage.

Soils classified as SC, SM, GC or GM, may be suitable as wall backfill if kept dry. Where drainage is adequate, theoretical earth pressures are valid for design purpose. These soils cannot be properly compacted when wet.

5.5.3 Residual soil

(with stones, fine silty sand, and granular mixtures with conspicuous clay content)

This type of soil is typically found inland, a nonhomogeneous mixture of ordinary fill materials that is used primarily because of ready availability and cost. This fill is more likely to be found insitu at land-sited walls. Its shear strength properties may be determined by carrying out tests on representative disturbed soil samples correlating the compaction states of backfill at site. Insitu density of backfill can be determined by sand replacement method or using nuclear density cone penetrometer (ND-CP) to verify the design parameters during construction stage.

5.5.4 Silts and Clayey Silts

Soils classified as CL, MH or ML are often subject to swelling when used as wall backfill and wall movement is likely to be excessive; the use of these materials should be avoided. This type of backfill is the type that should be excavated and disposed of as quickly as possible since it has none of the characteristics for permanent retaining structures. Quite often however, this material is encountered in building temporary cofferdams in which case it is acting as a load rather than part of an engineered structure. In other cases it might be found at the site of a permanent bulkhead and may be impractical to remove. In this case, special methods for improving the quality of this fill or ground improvement technique may be employed as part of the design.

5.5.5 Medium or Stiff Clay

Soils classified as CH, CL or OL are generally undesirable. Their long-term strength is difficult to access and then generally produces active earth pressure much higher than granular materials. They have poor drainage characteristics. Wherever it is inevitable ‘Sand Dikes’ may be used to provide a safe and optimal solution, which is briefed below.

5.6 Sand Dikes

When granular backfill is scarce, a sand dike may be place to form a plug across the potential failure surface of the active wedge as shown in Figure 5.2.

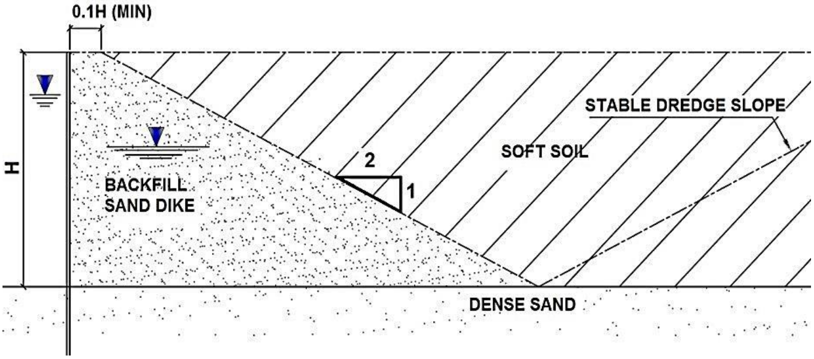


Figure 5.2 Sand Dike for controlling the Active Earth Pressure

Where such a dike rests on firm foundation soil, the lateral pressure on the bulkhead will be only the active pressure of the dike material. For further guidance, “Foundations Retaining and Earth Structures, by Tschebotarioff” may be referred. Tschebotarioff proposed the use of sand dikes adjacent to bulkhead walls that would provide the desirable characteristics of granular materials while permitting the use of poorer quality dredged material up-land from the wall. This method is commonly used.

While designing a permanent retaining system, the properties of backfill adopted in design and achieved during construction should be substantiated. Terzaghi recommended that “if a wall is designed before the backfill is selected, design for the worst condition, otherwise specify the characteristics of the backfill and design accordingly.” If backfilling is to be done after installation of wall or during construction in stages, following backfilling management process may be adopted.

5.7 Backfilling Management for Retaining Wall

5.7.1 Placement of Backfill

Backfilling of sheet pile walls may be initiated after a significant length of wall has been constructed. Ideally, all backfill should be clean and free draining unless accommodated in design. All backfill should be placed in horizontal lifts. For clean sands and gravels, the uncompacted lift thickness can be up to 30cm; for silts, clays, and silty clay mixtures, the maximum thickness should be 20cm.

5.7.2 Compaction of Filling Material

Compaction of dumped backfill may be desirable to reduce the materials compressibility, increase its strength, control expansion and reduce the post construction settlement. If the backfill is to be compacted, it should be mechanically compacted to 92 to 93 percent of the maximum dry density obtained by the Standard Proctor Test. For moisture sensitive soils, the water content should be controlled in the range of ± 2 percent of optimum moisture content. However, compaction will reduce the permeability and over-compaction can greatly increase lateral pressure near the top of the wall and unfavourably for the anchorage. Increasing the density of soil in the passive area and anchor wall area is desirable if it can be accomplished. However, compaction on the active side should be carefully specified both in the degree of densification and the method of attainment. Heavy compaction equipment should be kept well back from newly constructed bulkheads. Compaction within 2 meters of the back of the wall shall be accomplished with lightweight hand operated compaction equipment. If specified, densification can be accounted for in assigning engineering values to the soil for design. Hydraulic fill cannot be compacted during placement. Hydraulic fill should be accounted for in design as a temporary, increased load against a possibly unstabilised wall.

5.7.3 Effects of Backfill Extent

When designing a retaining wall for a design load based on active pressure, the design should take into consideration the extent of the backfill. The maximum active pressure may develop either from limiting conditions occurring entirely within the backfill material or from potential failure surfaces within the material present behind the zone of backfill. Therefore, to anticipate the lateral earth pressures correctly, the designer should anticipate the construction methodology and wall/soil placement technique, the direction of wall movement relative to the soil, and the probable worst groundwater conditions. In general, the designer should have full overview of followings along the proposed alignment of wall;

i) **Geometry of the existing site** — height and inclination of supported soil face, geometry of existing ground surface, distribution of soil types in three dimensions, position of any external loads, and position of any existing structures in the vicinity to be protected.

ii) **Bulk density of the soil**—this will often be in the range 16–22 KN/m^3, and may well increase with depth.

iii) **Groundwater conditions**— it is essentially required, in order to evaluate the final groundwater level, and pore water pressure variations on the shear surface and the back of the wall. It may also be necessary to carry out seepage studies.

iv) **Shear strength parameters**— In general, the designer will require information of sub-soil stratification and effective shear strength parameters (c', ϕ') for the soil that the wall is supposed to support.

v) **Soil stiffness parameters**—details of soil stiffness are required to analyses a wall and ground displacements using finite difference or finite element method. Soil stiffness values can be obtained from *in situ* tests, from laboratory tests, or from back-analysis of similar structures in similar ground conditions.

5.8 Effect of Wall movements on Earth Pressure

The amount of movement required to develop minimum active or maximum passive earth pressures depends on the stiffness of the soil and the height of the wall. For stiff soils like dense sands or heavily over-consolidated clays, the required movement is relatively small. Figure 5.3 illustrates the dominant role of soil strain in determining the horizontal stress acting on the supporting structure.

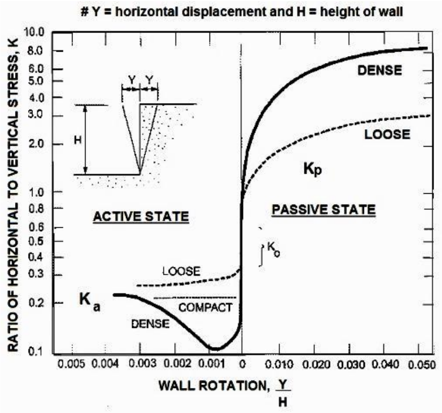


Figure 5.3 Effect of deformation on earth pressures in cohesionless materials

The magnitude of wall rotation, Y / *H,* required to achieve active and passive earth pressure conditions in various soil types is indicated in Table 5.1.

Table 5.1 *Effect of deformation on earth pressures in cohesionless materials*

Soil Type and Condition	Rotation, Y/H	
	Active	Passive

Dense cohesionless	0.001	0.02
Loose cohesionless	0.004	0.06
Stiff cohesive	0.010	0.02
Soft cohesive	0.020	0.04

Where, Y amount of horizontal translation or rotational displacement of the wall top relative to the wall bottom. Soft cohesive materials will continue to creep at pressures close to active and passive earth pressures and with time the deformations will increase if the walls are sufficiently flexible to allow full development of active and passive earth pressures.

The design procedures for retaining system described in this chapter assume that the sheet pile walls have sufficient flexibility to produce the limit state, active or passive earth pressures. For relatively rigid system, based on wall deformation an intermediate to extreme values of earth pressure may be adopted after soil-structure-interaction analysis as presented in next chapter.

5.9 Wall Friction and Adhesion

The relative vertical movement along the wall and soil interface may result in vertical shearing stresses due to wall/soil friction in the case of granular soils or in wall/soil adhesion for cohesive soils. This will have an effect on the magnitude of the minimum and maximum horizontal earth pressures.

In general, the effect of wall friction on active pressure is small and is often disregarded. The effect of wall friction on Coulomb's passive pressure is large, but definite movement is necessary for mobilization of wall friction. In the absence of specific test data, the angle of wall friction δ, where applicable, should be assumed to be in the range of one half to two thirds of ϕ. Values of δ greater than *1/2* of ϕ should not be used in calculations of passive earth pressure using the Coulomb approach since unrealistically large estimates will result. Values for friction factors between dissimilar materials are provided in Table 5.2.

TABLE 5.2 *Angles and Coefficients for Friction and Adhesion between Dissimilar Materials*

Structure Materials	**Ground or Backfill Materials**	**Interface Friction Angle, δ**	**Friction factor, tanδ**
Friction Materials			
Mass concrete or masonry	Clean sound rock	35	0.7
	GW, GP	29 to 31	0.55 to 0.60
	GM, GC, SW, SP	24 to 29	0.45 to 0.55
	SM, SC	19 to 24	0.35 to 0.45
	ML (non-plastic)	17 to 19	0.30 to 0.35
	CL (stiff to hard), residual soils	22 to 26	0.40 to 0.50
Steel sheet piles	Well graded rock fill	22	0.40
	Uniform size rock fill	17	0.30
	GW, GP	22	0.40
	GM, GC, SW, SP	14	0.25
	SM, SC	14	0.25
	ML (non-plastic)	11	0.20
Formed of pre-cast concrete	Well graded rock fill	22 to 26	0.40 to 0.50
	Uniform size rock fill	17 to 22	0.30 to 0.40
	GW, GP	22 to 26	0.40 to 0.50
	GM, GC, SW, SP	17	0.30
	SM, SC	14	0.25
	ML (non-plastic)	14	0.25
Adhesion in Cohesive Soils			
Consistency	Undrained Shear Strength, Su, KPa		Adhesion (KPa)
Very Soft	0 to 12		0 to 12
Soft	12 to 25		12 to 25
Medium Stiff	25 to 50		25 to 38
Stiff	50 to 100		38 to 48
Very Stiff	100 to 200		48 to 65

5.10 Earth Pressure Calculations

The design values of lateral earth pressure are intended to give an overestimate of the earth pressure on the active or retained side and an underestimate of the earth resistance on the passive side for small deformations of the structure as a whole, in the working state.

In the special case of a smooth vertical wall and horizontally retained soil surface (δ = 0, α = 90°, and β= 0), Rankine's formula may be used

$$K_a = (1 - \sin \emptyset)/(1 + \sin \emptyset) \tag{5.16}$$

Several other earth pressures theories are available for estimating the minimum (active) and maximum (passive) lateral earth pressures that can develop in a soil mass surrounding a wall and it should be applied judiciously according to the requirement and complexity of site.

5.10.1 Earth Pressure for Normal consolidated clay

For normal consolidated and lightly over-consolidated clay, the design undrained shear strength C_u can be used satisfactorily, because in a total stress analysis, the short-term condition is usually critical, except for excavation in front of embedded walls.

5.11 Lateral earth pressure due to surcharge Loads

Loads due to stockpiled material, machinery, roadways, and other influences resting on the soil surface in the vicinity of the wall increase the lateral pressures on the wall. The surcharge load shall be applied on the surface of retained soil during design of retaining walls. According to BS 8002, minimum value of surcharge load should be in order of 10 kN/m^2.

Additional surcharge loading should be used in the design to take care of incidental loads arising from construction plant, stockpiles of materials and traffic movement both during construction and subsequent operation.

The various surcharges imposed behind a wall may be classified as:

i) Uniform surcharge load
ii) Strip loads
iii) Line loads
iv) Point loads
v) Ramp load

vi) Triangular load
vii) Area loads

If the soil system admits to application of the coefficient method, the effects of surcharges, other than a uniform surcharge, are evaluated from the theory of elasticity solutions presented in the following paragraphs.

5.11.1 Uniform surcharge load

Uniform surcharge is a uniformly distributed load, consisting of a continuous load on the surface of the backup area behind quay walls, road embankment etc. It is assumed to be applied at all points on the soil surface. The effect of the uniform surcharge is to increase the effective vertical soil pressure, P_v in Equations 5.5 and 5.9, by an amount equal to the magnitude of the surcharge.

5.11.2 Strips loads.

A strip load is continuous loadings which may be due to strip footings of sheds, rail track etc. which may run parallel to the wall but is of finite extent perpendicular to the wall as illustrated in Figure 5-4. The additional pressure on the wall is given by the equations in Figure 5-4. Any negative pressures calculated for strips loads are to be ignored.

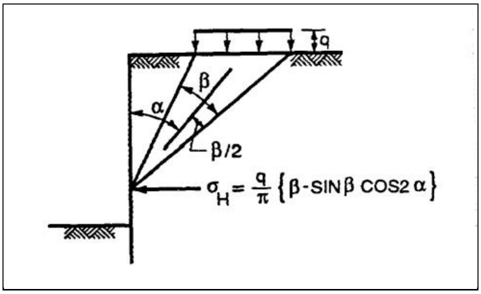

q
β
α
β/2
$\sigma_H = \frac{q}{\pi}\{\beta - \sin\beta \cos 2\alpha\}$

Figure 5-4 Strip Load

5.11.3 Line loads

A continuous load parallel to the wall but of narrow dimension perpendicular to the wall may be treated as a line load as shown in **Figure 5-5**. The lateral pressure on the wall is given by the equation in **Figure 5-5**.

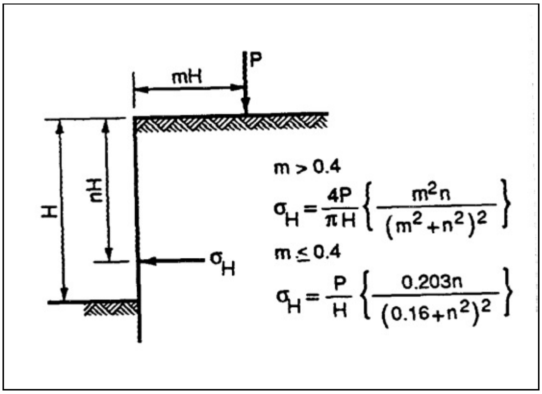

P
mH
H
nH
σ_H
m > 0.4
$\sigma_H = \frac{4P}{\pi H}\left\{\frac{m^2 n}{(m^2+n^2)^2}\right\}$
m ≤ 0.4
$\sigma_H = \frac{P}{H}\left\{\frac{0.203n}{(0.16+n^2)^2}\right\}$

Figure 5.5 Line load

5.11.4 Ramp load

A ramp load, Figure 5-6, increases linearly from zero to a maximum which subsequently remains uniform away from the wall. The ramp load is assumed to be continuous parallel to the wall. The equation for lateral pressure is given by the equation in Figure 5-6.

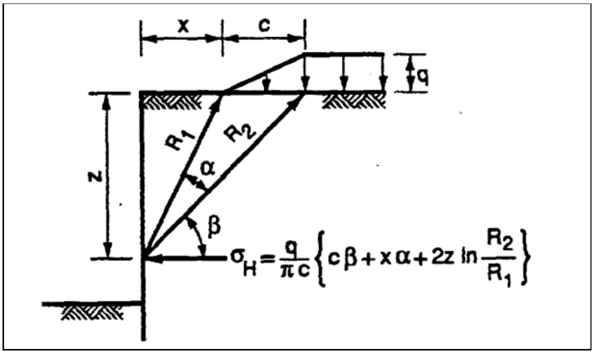

x
c
q
z
R_1
R_2
α
β
$\sigma_H = \frac{q}{\pi c}\left\{c\beta + x\alpha + 2z \ln\frac{R_2}{R_1}\right\}$

Figure 5.6 Ramp load

5.11.5 Triangular loads

A triangular load varies perpendicular to the wall as shown in Figure 5-7 and is assumed to be continuous parallel to the wall. The equation for lateral pressure is given in Figure 5-7.

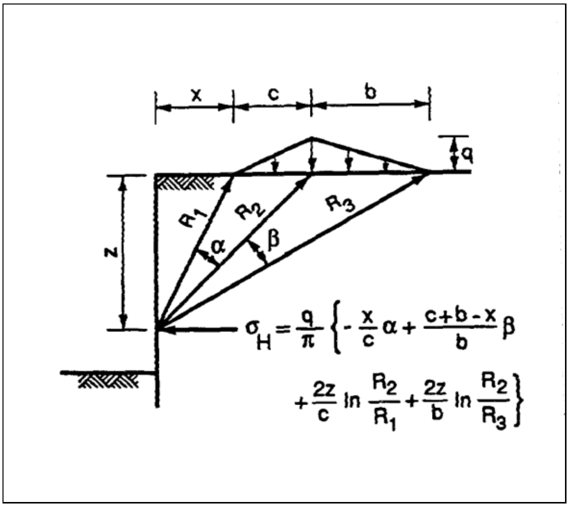

x
c
b
q
z
R1
R2
R3
α
β
$\sigma_H = \frac{q}{\pi}\left\{-\frac{x}{c}\alpha + \frac{c+b-x}{b}\beta + \frac{2z}{c}\ln\frac{R_2}{R_1} + \frac{2z}{b}\ln\frac{R_2}{R_3}\right\}$

Figure 5.7 Triangular load

5.11.6 Area loads

A surcharge distributed over a limited area, both parallel and perpendicular to the wall, should be treated as an area load. The lateral pressures induced by area loads may be calculated using Newmark's Influence Charts (Newmark 1942). The lateral pressures due to area loads vary with depth below the ground surface and with horizontal distance parallel to the wall. Because the design procedures discussed subsequently are based on a typical unit slice of the wall/soil system, it may be necessary to consider several slices in the vicinity of the area load.

5.11.7 Point loads

A surcharge load distributed over a small area may be treated as a point load e.g., concentrated loads from isolated footings etc. The equations for evaluating lateral pressures are given in Figure 5-8.

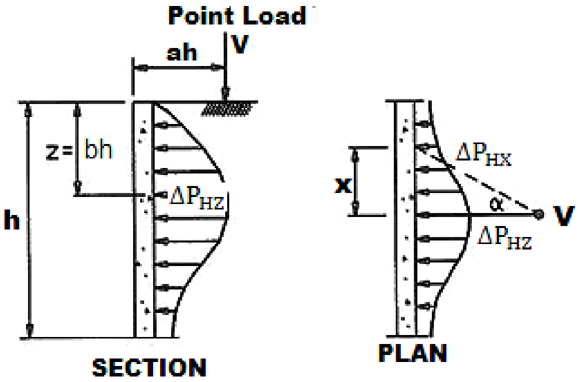


Figure 5.8 Lateral Earth Pressure due to Point Load

Increase in horizontal pressure, ΔP_{HZ}, on a section through point load, V

$$\Delta P_{HZ} = \left(\frac{V}{h^2}\right)\left(\frac{a^2 b^2}{(a^2 + b^2)^3}\right) \qquad for\ a > 0.4 \qquad (5.17)$$

$$\Delta P_{HZ} = \left(\frac{0.28V}{h^2}\right)\left(\frac{b^2}{(0.16 + b^2)^3}\right) \quad for\ a \le 0.4 \qquad (5.18)$$

Increase in horizontal pressure, ΔP_{HZ}, at distance X from plane of load, V

$$\alpha = tan^{-1}\left(\frac{X}{ah}\right)$$

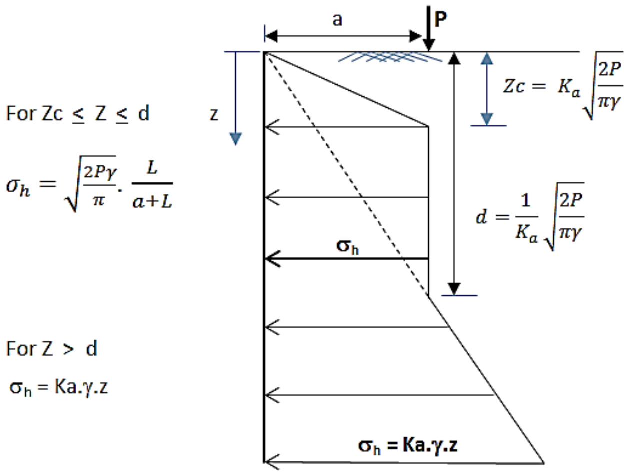
$$\Delta P_{HX} = \Delta P_{HZ}\, cos^2(1.1\alpha) \qquad (5.19)$$

Because the pressures vary horizontally parallel to the wall; it may be necessary to consider several unit slices of the wall/soil system for design.

5.11.8 Compaction-Induced Pressures

Compaction of backfill behind retaining structures can induce loads greater than the active or at-rest earth pressures estimated using Coulomb or Rankine earth pressure theory. Compaction-induced pressures can be such that the structural capacity of walls designed to withstand K_o or Ka can be exceeded. Figure 5.9 illustrates the general distribution and calculation of stresses from compaction.

In general, for non-yielding walls where activity by mechanical compaction equipment is anticipated within a distance of one-half the height of the wall, the effect of additional earth pressure that may be induced by compaction shall be taken into account.


a
P
z
For Zc ≤ Z ≤ d
$\sigma_h = \sqrt{\frac{2P\gamma}{\pi}} \cdot \frac{L}{a+L}$
$Zc = K_a \sqrt{\frac{2P}{\pi\gamma}}$
$d = \frac{1}{K_a}\sqrt{\frac{2P}{\pi\gamma}}$
σ_h
For Z > d
$\sigma_h = Ka.\gamma.z$
$\sigma_h = Ka.\gamma.z$

$$P\ (Roller\ Load) = \frac{Dead\ weight\ of\ roller + Centrifugal\ force}{Weight\ of\ roller}$$

a = Distance of roller from wall; L = Length of roller;

Figure 5.9 Horizontal Pressures on Walls from Compaction Effort

5.12 Water Loads

5.12.1 Hydrostatic pressure

A difference in water level on either side of the wall creates an unbalanced hydrostatic pressure. Water pressures are calculated by multiplying the water depth by its specific weight. When wall is intended to prevent all leakage of groundwater, the unbalanced hydrostatic pressure is assumed to act along the entire depth of embedment. Water pressure must be added to the effective soil pressures to obtain total pressures.

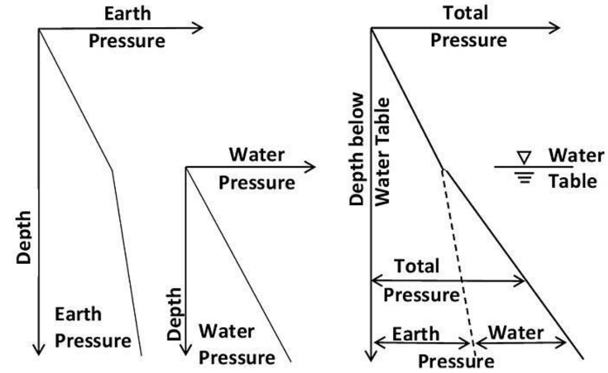

Earth Pressure
Depth
Earth Pressure
Water Pressure
Depth
Water Pressure
Total Pressure
Depth below Water Table
Water Table
Total Pressure
Earth
Water
Pressure

Figure 5.10 Lateral Pressures due to Ground Water Table

Maritime structures are subject to the effects of tidal variations, and storm runoff. The design should take account of the possible build-up of ground water levels behind the wall. To assess the design level of water table, natural variations in the ground water table, and the provision of effective drainage and the drainage characteristics of backfill and drainage layers provided should be taken into the account. The influence of rainstorms on the seepage pattern behind the structure should also be considered, particularly for sandy silts and silts.

5.12.2 Wave action

The lateral forces produced by wave action are dependent on many factors, such as length, height, breaking point, frequency and depth at structure. Wave forces for a range of possible water levels should be determined in accordance with the U.S. Army Coastal Engineering Research Centre Shore Protection Manual.

5.13 Mooring pulls

Lateral loads applied by a moored ship are dependent on the shape and orientation of the vessel, the wind pressure, and currents applied. Due to the use of strong synthetic lines, large forces can be developed. Therefore, it is desired that mooring devices be designed independent of the sheet pile wall or anchorage system should be efficient enough to bear the mooring pull without affecting the sheet pile walls stability.

5.14 Wind forces

When sheet pile walls are constructed in exposed areas, wind forces should be considered during construction and throughout the life of the structure. For more sever conditions, wind load should be computed in accordance with the relevant Standard specified for the project.

5.15 Earthquake forces

Earthquake forces should be considered in the zones of seismic activity. The earth pressures should be determined in accordance with the applicable standards for the project. In the worst case, the supporting soil may liquefy allowing the unsupported wall to fail. This possibility should be evaluated and addressed during design and stability check. If accepting the risk and consequences of a liquefaction failure is unacceptable, consideration should be given to replacing or improving the liquefiable material or alternatively, relocating the wall.

5.16 Additional Applied Loads

Sheet Pile walls are widely used in many applications and can be subjected to a number of additional loads, other than lateral pressure exerted by soil and water.

Chapter 6

Analysis for System Stability

Engineering is a knowledge of the mathematical and natural sciences gained by study, experience, and practice is applied with judgment to develop ways to utilize, economically, the materials and forces of nature for the benefit of mankind.

- *ABET*

6.1 Introduction

Stability analysis of structure is the process to determine the response of a structure under applied loading conditions or combinations of various loads. The principal goal in the design of any general engineering structure is to achieve stability and it must remain stable and fit throughout its useful life. To carry out appropriate analysis and design, engineer has to simplify and to idealise the structure according to real world existence. ***"To analyse is first to idealise"***.

Assessment of stability of retaining structure must include a rational assessment of loads and must account for the basic structural behaviour, the mechanism of transmitting lateral loads to the supporting soil, the subgrade pressures and the secondary effects of the soil deformation on the structure. Under static conditions, retaining walls are acted upon by lateral earth pressures, water pressures and other external forces. A properly designed retaining wall will achieve equilibrium of these forces without inducing unbalanced shear stresses into the supporting soil medium. Failure of retaining walls, whether by sliding, tilting, bending, or some other mechanism, occurs when these permanent deformations become excessive in soil-structure support system. The level of deformation depends on many factors and is best addressed on the basis of serviceability requirements of the system.

For an adequate stability assessment, it is necessary to define "mode of failures" and to know the probable mechanism how walls can fail. The loads exerted on wall/soil system tend to produce a variety of potential failure modes. These failure modes and selection of certain system parameters to prevent failure are discussed in this chapter. The evaluations of the loads on the system are already discussed in previous chapter.

6.2 Stability Considerations

Braced walls usually fail by gross instability, tilting, flexural failure, and/or failure of bracing elements. Tilting of braced walls typically involves rotation about the point at which the brace acts on the wall. Anchored walls with inadequate penetration may tilt by "pushing out" at their toes. As in the case of cantilever walls, anchored walls may fail in flexure, although the point of failure (maximum bending moment) is likely to be different. Failure of anchorage elements can include anchor pull-out, tie-rod failure. Backfill settlements can also impose additional axial and transverse loading on bracing elements such as tie-rods and tiebacks.

6.3 Modes of Failure for Anchored Wall

Anchored sheet pile wall have multiple potential failure mechanism. The mode of failures presented by BS Code and US Army Corps of Engineers are as follows.

6.3.1 Deep-seated failure

A deep-seated soil failure is a large-scale movement of the soil mass on both sides of the wall. As noted by the Army Corps of Engineers, it is “independent of the structural characteristics of the wall” and is entirely due to the resistance of the soil mass to shearing. A potential rotational failure of an entire soil mass containing an anchored or cantilever wall is illustrated in Figure 6-1. This type of failure is independent of the structural characteristics of the wall and/or anchor. The adequacy of the system (i.e. factor of safety) against this mode of failure should be assessed by conventional slope stability analysis.

This type of failure cannot be remedied by increasing the depth of penetration or by repositioning the anchor. The only recourse when this type of failure is anticipated is to change the geometry of retained material or improve the soil strengths.

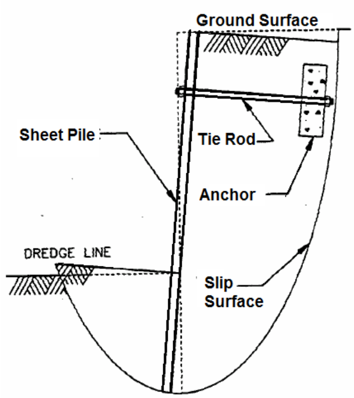

Ground Surface
Sheet Pile
Tie Rod
Anchor
DREDGE LINE
Slip Surface

Figure 6.1 – Deep Seated Failure Mechanism for Vertical Retaining Wall

6.3.2 Rotational Failure (Toe Failure)

During a toe failure, the soil mass in front of the wall does not provide enough lateral resistance to resist the load applied due to the soil and surcharge loadings behind the wall. This type of failure is prevented by adequate penetration of the piling in a cantilever wall or by a proper combination of penetration and anchor position for an anchored wall.

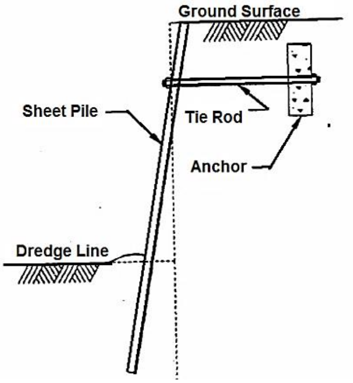

Ground Surface
Sheet Pile
Tie Rod
Anchor
Dredge Line

Figure 6.2 – Rotational Failure Mechanism for Anchored Sheet Pile Wall

6.3.3 Wale System Failure

A wale system failure is a separation of the anchor wall from the waling beam. This type of failure mechanism is a function of the loading conditions and connection detailing, which shall be taken care adequately during detail design.

The waling should be designed as a beam, supported on elastic supports provided by sheet piling both in horizontal and vertical directions. If waling beam consists of RCC, sufficient reinforcement should be provided in the capping beam to cope with shear forces arising from anchor head loads and also with shrinkage and temperature stresses.

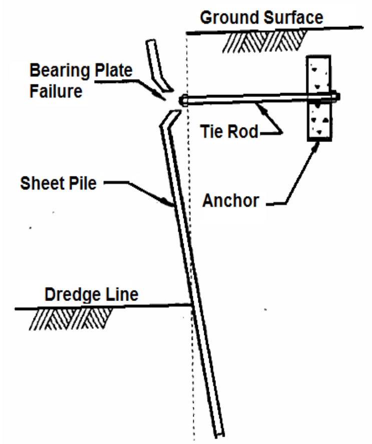

Ground Surface
Bearing Plate Failure
Tie Rod
Sheet Pile
Anchor
Dredge Line

Figure 6.3 Wale System Failure Mechanism for vertical Retaining Wall

6.3.4 Anchor Failure

Anchor failure is the large movement of anchorage system. This type of failure mechanism is a function of soil strength, geometry and loading conditions.

In order for an anchorage system to be effective it must be located outside the potential active failure zone developed behind a sheet pile wall. Its capacity is also impaired if it is located in unstable ground or if the active failure zone prevents the development of full passive resistance of the system. The lateral pull capacity of anchor wall should be analysed by conventional means, considering

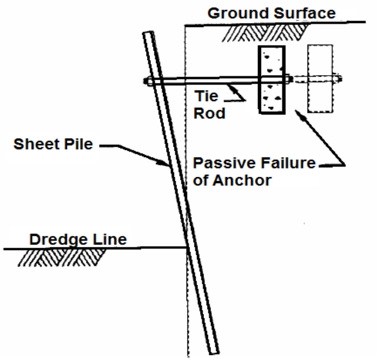

Ground Surface
Tie Rod
Sheet Pile
Passive Failure of Anchor
Dredge Line

Figure 6.4 Anchor Failure Mechanism for Wall

full passive pressure of surrounded soil, only if active and passive failure zones do not intersect.

6.3.5 Tie rods Failure

Where tie rods are employed, they may be critical to the stability of sheet piled walls. The design should have provision for the increase in stresses which may arise from corrosion and also through the bending of tie rods due to the ground around them settling. For protection against corrosion buried permanent tie rods should be given a suitable encapsulation, wrapping or coating, sufficiently flexible to accommodate the extension of the tie rods under load. Provision should be made in addition for corrosion, if the ties are placed in aggressive ground or sea water.

The tie rods should be provided with washers and bearing plates to give adequate bearing on the waling or deadman (anchor blocks). Tie rods having lengths over 12 m should be joined by couplers or turnbuckles. The rod diameter at the root of the threads of nuts and turnbuckles should be adequate to take the design tie load. If settlement of the soil below tie rods is likely to occur, then the tie rods may be enclosed in flexible plastic ducts.

6.3.6 Flexural Failure of Sheet Pile

Flexural failure of sheet pile wall occurs when the stress in the main structure of the pilings is high enough to develop a plastic hinge. This type of failure mechanism is a function of the loading conditions, and structural design taking care of deterioration due to corrosion in marine environment.

During flexural failure of piling, the stress in sheet piles are such that at a particular location the entire cross section has reached the ultimate capacity steel. At this point displacement of the sheet pile will increase without a corresponding increase in loading. This condition is referred to a plastic hinge because the response of the sheet pile behaves similarly to a beam with a hinge. Depending upon the embedment, this hinge may lead to instability of the wall.

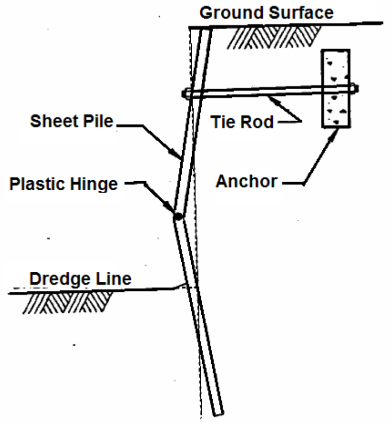


Figure 6.5 Flexure Failure Mechanism for Anchored Sheet Pile Wall

6.3.7 Rotational Failure Due to Inadequate Penetration

(Toe Failure)

Rotational failure due to inadequate penetration is a large-scale movement of the lower portion of the wall; commonly referred to as a toe failure. This type of failure mechanism is a function of the loading conditions, soil strength, and wall embedment. During a toe failure, the soil mass in front of the wall does not provide enough lateral resistance to resist the load applied due to the soil and surcharge loadings behind the wall.

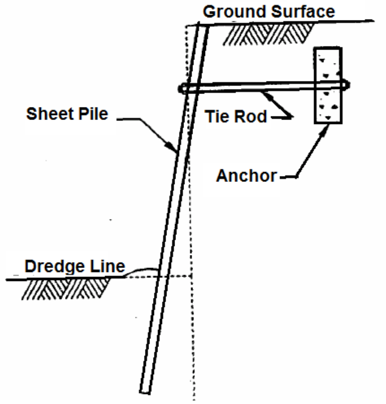


Figure 6.6 Rotational Failure Mechanism for Anchored Sheet Pile Wall

6.4 Modes of Failure for Wall with Ground Anchor

Based on previous discussion, the possible modes of failure for tied-back retaining walls (wall with ground anchor) are provided below.

6.4.1 Tensile Failure of Tendon

As the anchor is loaded, the steel tendon component of the anchor is stressed in tension. If the applied load is greater than the structural capacity of the tendon, failure is inevitable. Therefore, a factor of safety must be used with respect to structural failure of the steel. It is recommended that the tendon load not exceed 60 percent of the specified minimum tensile strength (SMTS) for final design and 80 percent of SMTS for temporary loading conditions (e.g., loading during testing).

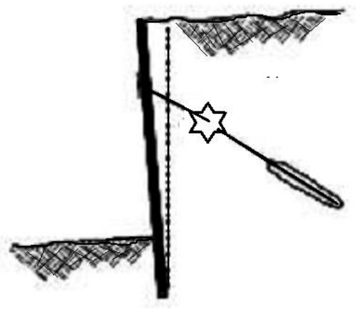


Figure 6.7 Tensile Failure of Tendon in Anchored Wall

6.4.2 Grout-Ground Bond Failure

Failure of the grout-ground bond involves, failure resulting from anchor loads, not external forces such as landslides which potentially introduce excessive static loading to the anchor. A shear surface develops in the soil mass ahead of the anchor as increasing stresses cause complete mobilization of soil in the anchor bond zone. Generally, this should always be established on site by proof testing given the difficulty in predicting the capacity and the dependence on installation technique.

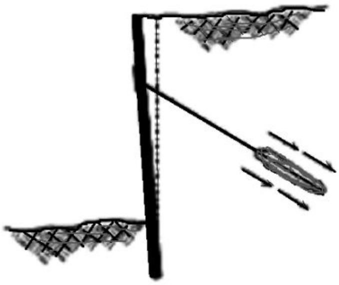


Figure 6.8 Pull out failure of Grout-ground bond failure

6.4.3 Tendon-Grout Bond Failure

The bond between the grout and steel tendon must not be exceeded if the full strength of the supporting ground is to be mobilized.

The bond resistance between grout and steel is a function of compressive strength of grout, which increases with increase in compressive strength of the grout.

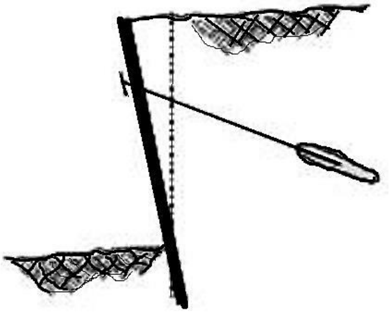


Figure 6.9 Pull out failure of Tendon-grout bond failure

6.4.4 Wall bending failure

It may happen due to flexural failure or buckling failure of wall structure, when the stress is high enough to develop a plastic hinge.

Actual wall bending moments should not exceed the design capacity during service, taking care of section loss due to corrosion.

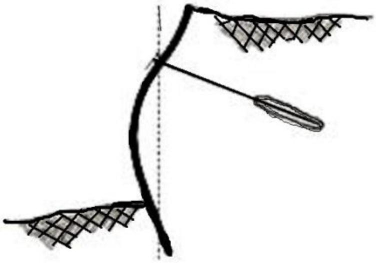


Figure 6.10 Failure due to bending in wall

6.4.5 Basal failure of anchored wall

Insufficient embedment depth for wall or progressive strain softening of underlying clayey strata leads to passive failure of the soil immediately in front of the wall to cause instability of the wall and soil mass.

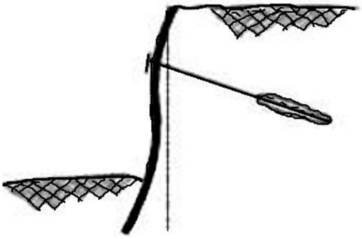


Figure 6.11 Basal failure of anchored wall

6.4.6 Forward rotation of wall

This type of failure may happen before installation of ground anchors. Staging of excavation is necessary to prevent forward rotation of wall prior to anchor installation. Wall needs sufficient bending strength to resist cantilever moments during construction stage.

Also, structural and geotechnical capacity of anchors should be of sufficient capacity and length to prevent forward rotation.

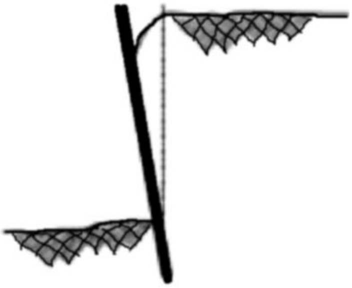


Figure 6.12 Failure due to forward rotation of wall

6.5 Verification for Overall Stability of Anchored Systems

Even though appropriate loads, pressures and anchor design criteria are satisfactory, an earth retaining structure supported by anchors can fail if the entire block encompassing all wall components is not stable. Therefore, the walls tied back with highly stressed ground anchors or Tie rods, the stability of wall shall be verified by applying additional stability checks.

The verification of the stability consists of the verification of the possible unfavourable and deep sliding planes (Bishop method), the stability of the anchorage (Kranz method), the verification of heave of the stabilizing soil layer on the passive side of the sheet pile, a phenomenon called piping and local buckling in steel sheet pile wall.

6.5.1 Bishop method

The method of Bishop is based on stability of a in circular sliding plane as shown in Figure 6.13. Where the mode of failure involves a rotational or translational movement, the stable equilibrium of the earth retaining structure depends on the mobilization of shear stresses within the soil. The full mobilization of the soil shear strength gives rise to limiting active and passive thrusts.

For a stable system, the destabilizing moment (M_O) has to be smaller or equal than the resisting moment (M_R).

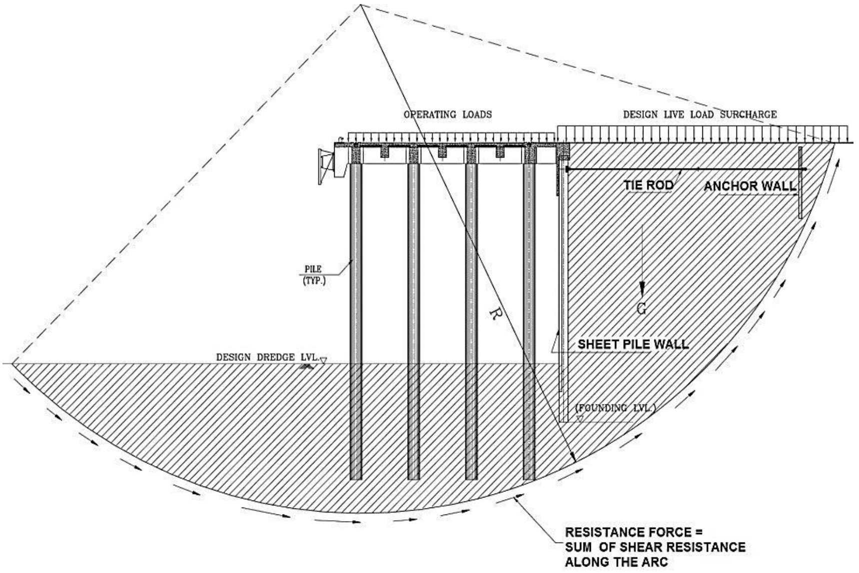

OPERATING LOADS
DESIGN LIVE LOAD SURCHARGE
TIE ROD
ANCHOR WALL
PILE (TYP.)
R
G
SHEET PILE WALL
DESIGN DREDGE LVL.
(FOUNDING LVL.)
RESISTANCE FORCE = SUM OF SHEAR RESISTANCE ALONG THE ARC

Figure 6.13 Slope stability analysis of wall with tie rods

Guidelines advise to apply a ratio between the destabilizing and resisting moment (FS > 1.5, which is the overall safety factor regarding stability of a wall). For the calculation of these moments, conventional slope stability analysis is made for most unfavourable sliding plane.

6.5.2 Kranz method

A check on overall equilibrium ensures that tie forces are transmitted far enough back from the wall (i.e. that the free length of anchors is adequate). The method proposed by Kranz (1953) examines the equilibrium of a wedge of soil between the back of the wall and the front of the fixed length. Considering the forces acting on the wedge, a force polygon may be constructed to determine the maximum anchor force to just maintain equilibrium.

The Kranz method is used to verify the stability of the anchorage; this is based on a deep straight sliding plane. If tension piles are used, the sliding plane goes from the shear centre of the sheet piling to the centre of the anchorage area of the tension piles. If an anchorage with anchor wall is used, the slide plane goes from the shear centre of the wall to the toe of the anchor wall. The groundmass in the sliding plane is loaded with the design value of the tensile force of anchorage or pile and must be in equilibrium.

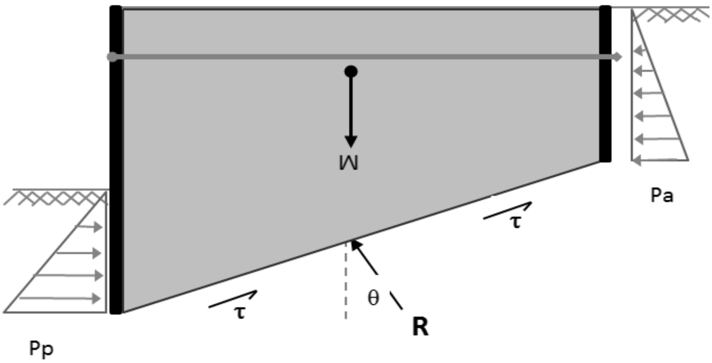


Figure 6.14 Stability analysis of wall with tie rods (Kranz Method)

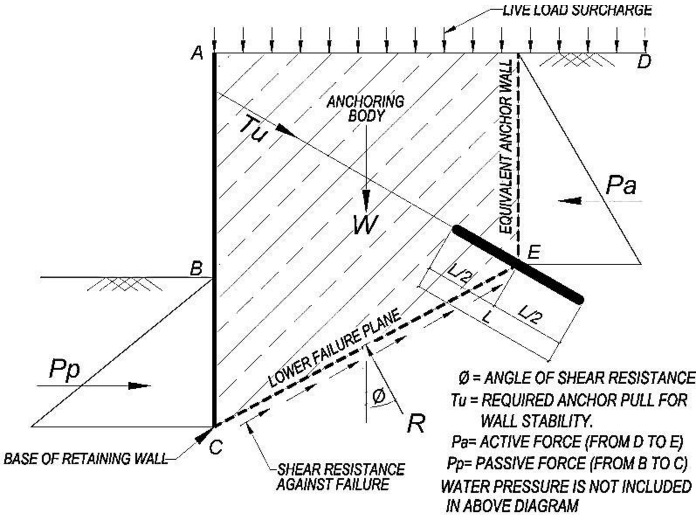

LIVE LOAD SURCHARGE
A
D
ANCHORING BODY
Tu
EQUIVALENT ANCHOR WALL
Pa
W
E
B
L/2
L
L/2
LOWER FAILURE PLANE
Pp
Ø
R
C
BASE OF RETAINING WALL
SHEAR RESISTANCE AGAINST FAILURE
Ø = ANGLE OF SHEAR RESISTANCE
Tu = REQUIRED ANCHOR PULL FOR WALL STABILITY.
Pa= ACTIVE FORCE (FROM D TO E)
Pp= PASSIVE FORCE (FROM B TO C)
WATER PRESSURE IS NOT INCLUDED IN ABOVE DIAGRAM

Figure 6.15 Slope stability analysis of wall with Ground anchors (Kranz Method)

A factor of safety is defined as follows:

$$F = \frac{T_{max}}{T_{applied}} \geq 1.5$$

A local equilibrium check ensures that forces transmitted to the soil do not cause local failure of the soil; i.e. that the fixed length is adequate. Broms (1968) suggests considering the equilibrium of a wedge of soil above the fixed length. If the forces are not in equilibrium, the fixed length must be increased.

6.6 Load cases for Stability Analysis

The loads applied to a wall fluctuate during its service life. Consequently, several loading conditions must be defined within the context of the primary function of the wall. The load cases outlined in project specification or relevant design standards should be considered in the design. Stability should be assessed according to following load conditions.

6.6.1 Usual conditions

The loads associated with this condition are those most frequently experienced by the system in performing its primary function throughout its service life. The loads may be of a long-term sustained nature or of an intermittent, but repetitive, nature. The fundamental design of the system should be optimized for these loads. Conservative factors of safety should be employed for this condition.

6.6.2 Unusual conditions

Construction and/or maintenance operations may produce loads of infrequent occurrence and are short duration which exceeds those of the usual condition. Wherever possible, the sequence of operations should be specified to limit the magnitudes and duration of loading, and the performance of the wall should be carefully monitored to prevent permanent damage. Lower factors of safety or higher material stresses may be used for these conditions with the intent that the system should experience no more than cosmetic damage.

6.7 Factor of Safety Required for Stability

As part of the design, the overall stability of the retaining wall, retained slope and foundation soil or rock shall be evaluated for all walls using limiting equilibrium methods of analysis. A minimum factor of safety of 1.3 shall be used for the design of walls for static loads, except that a minimum factor of safety of 1.5 shall be used for the design of walls which support waterfront structures, bulkheads, bridge abutments, buildings, critical utilities, or other installations for which there is a low tolerance for failure. A minimum factor of safety of 1.1 shall be used for the design of walls for seismic loads. In all cases, the subsurface conditions and soil/rock properties of the wall site shall be adequately characterized through in-situ exploration and testing and /or laboratory testing.

Example 6.1

A retaining wall is anchored with a deadman through tie rods as shown in ***Figure-E6.1****. Verify the translational stability of overall wall system in accordance with Kranz method. The dimensional details of wall and soil properties are given below.*

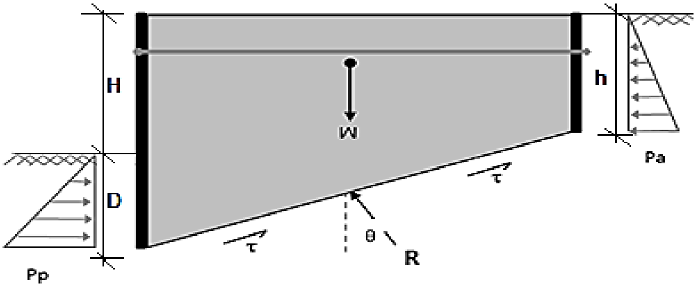


Figure E-6.1

Ht. of wall above mud line (H) = 8.5 m
Depth of wall below mud line (D) = 5.0 m
Depth of anchor wall (h) = 3.0 m
Width of Bulkhead (L) = 12.0 m
Bulk density of soil (g) = 18.0 KN/m^3
Angle of internal friction of soil at base (φ) = 33.0 deg.

Live Load Surcharge on bulkhead = 25.0 KN/m^2

Solution:

Total wt. of soil in bulkhead (W) = 1782 KN/m length

Live load = 300 KN/m length

Total Vertical Load (W_1) = 2082 KN/m length

From figure above;

Destablising Force (F_D) = $W_1 . \sin\theta + Pa$

= $W_1 . \sin\theta + Ka.g.h^2$

Resisting Force (F_R) = $W_1 . \cos\theta \, . \tan\varphi + Pp$

= $W_1 . \cos\theta \, .\tan\varphi + Kp.\gamma.D^2$

$\tan\theta = (H+D-h)/L$

$Ka = 0.29$

$Kp = 3.39$

$Pa = (1/2).Ka.\gamma.h^2$ = 47.76 KN/m length

$Pp = (1/2). Kp. \gamma.D^2$ = 1526.45 KN/m length

$\tan\theta = 0.875$

$\theta = 41.19$ deg.

$\sin\theta = 0.659$

$\cos\theta = 0.753$

$F_D = W_1 . \sin\theta + Pa$ = 1418.8 KN

$F_R = W_1 . \cos\theta . \tan\varphi + Pp$ = 2544.0 KN

F.O.S against sliding $= F_R / F_D = 1.79 > 1.5$

(O.K)

Example 6.2

A retaining wall is anchored with a tie-back as shown in ***Figure-E6.2****.* *Verify the translational stability of overall wall system in accordance with*

Kranz method. The dimensional details of wall and properties of soil are given below.

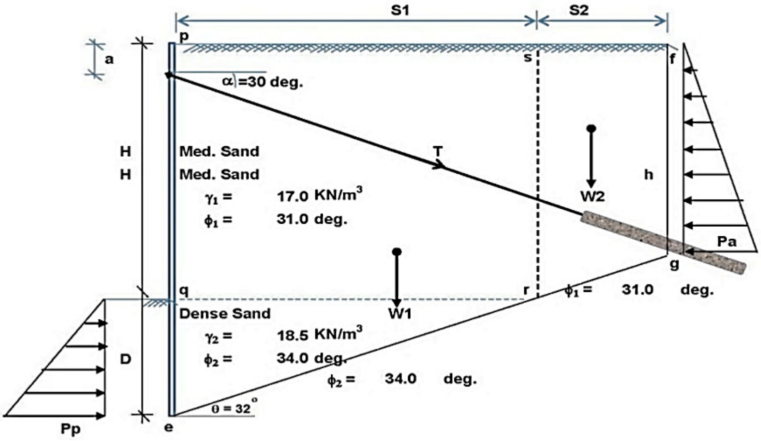

S1
S2
p
a
s
f
α=30 deg.
H
H
Med. Sand
Med. Sand
T
h
W2
γ1 = 17.0 KN/m3
ϕ1 = 31.0 deg.
Pa
g
q
r
ϕ1 = 31.0 deg.
Dense Sand
W1
γ2 = 18.5 KN/m3
ϕ2 = 34.0 deg.
D
ϕ2 = 34.0 deg.
θ = 32°
Pp
e

Figure E-6.2

Ht. of wall above mud line	*(H)*	=	*16.0*	*m*
Depth of wall below mud line	*(D)*	=	*9.0*	*m*
Virtual anchor wall (f-g) *(at C.G of Anchor)*	*(h)*	=	*13.5*	*m*
Depth of anchor head in wall	*(a)*	=	*2.0*	*m*
Anchor Pull at wall head	*(T)*	=	*300.0*	*KN/m*
Inclination of anchor	*(α)*	=	*30.0*	*deg.*

Solution:

Properties of Soil Layer-1

Type of soil - Medium Dense Sand

Bulk density of soil	(γ_1)	=	17.0	KN/m^3
Angle of internal friction of soil at base	(ϕ_1)	=	31.0	deg.
Width of soil wedge p-q-r-s	S1	=	15.6	m
Depth of soil wedge (p-q = r-s)		=	16.0	m

Properties of Soil Layer-2

Type of soil - Dense Sand

Bulk density of soil (γ_2) = 18.5 KN/m^3

Angle of internal friction (ϕ_2) = 34.0 deg.

Width of soil wedge f-g-r-s S2 = 4.4 m

Live Load Surcharge on bulkhead = 25.0 KN/m^2

Wt. of soil wedge p-q-r-s = 4243.2 KN/m length

Wt. of soil wedge e-q-r = 1298.7 KN/m length

LL on soil wedge p-q-r-s = 390.0 KN/m length

Wt. of soil wedge (W1) = 5931.9 KN/m length

Wt. of soil wedge f-g-r-s = 1200.65 KN/m length

LL on soil wedge f-g-r-s = 110.0 KN/m length

(W2) = 1310.7 KN/m length

Ka = 0.32

$Pa = 0.5.Ka.\gamma.h^2$ = 991.75 KN/m length

$\tan\phi_1$ = 0.601

Kp = 3.54

$Pp = 0.5.Kp.\gamma.D^2$ = 5300.39 KN/m length

$\tan\phi_2$ = 0.675

From figure above;

Slope of lower failure plane θ = 32.0 deg.

Destablising Force (F_D) = (W1+W2) $\sin\theta$ + Pa

(W1+W2) $\sin\theta$ = 3837.97 KN/m length

(F_D) = 4829.71 KN/m length

Resisting Force (F_R) = W1. $\cos\theta$. $\tan\phi_2$
+ W2. $\cos\theta$. $\tan\phi_1$
+ Pp + T.$\cos\alpha$

W1. $\cos\theta$. $\tan\phi_2$ = 3393.14 KN

W2. $\cos\theta$. $\tan\phi_1$ = 667.9 KN

T.$\cos\alpha$ = 259.8 KN

(F_R) = 9621.19 KN

F.O.S against sliding = F_R / F_D = 1.99 > 1.5 **(O.K)**

6.8 Special Cases to enhance the stability of Wall

a) Application of Multiple Anchors

Anchored Wall with Single Level of anchorage, is the usual form of bulkhead. However, more than one set of anchors or tiebacks can be used. It increases wall stability and enables taller walls to be built and sustained almost a necessity with sheet piles. Where variation in water level between HFL and LWL, especially in river front is large, multiple tie-rods can be used to optimise the front sheet pile wall section.

b) Mitigation of Settlement around the Wall

Care must be exercised to see that the anchor block or deadman does not settle after construction. This is generally not a problem in undisturbed soils, however, where the anchorage must be located in unconsolidated fill, piles may be needed to support the blocks. Also, the soil within the passive wedge of the anchorage should be compacted to at least 90 per cent of maximum density unless the deadman is forced against firm natural soil

6.9 Settlement Effects on Tie Rods

Tie rods placed above loose granular or soft cohesive soils can be subjected to loads greater than that computed by conventional methods.

- ❖ As the underlying soils compress, either due to volume changes, distortion, or consolidation, the weight of the overlying soils induces additional loads as the rod deflects. Where excavation is necessary to place an anchor, the backfilled material should be a selected soil, compacted to at least 90 percent of standard proctor maximum dry density. If soil conditions warrant the consideration of settlement, methods used in eliminating the effects include supporting the tie rod or encasing it in conduit

- Any soft soil below the tie rods, even at great depth, may consolidate under the weight of recent backfill, causing the ground to settle. A small settlement will cause the tie rods to sag under the weight of the soil above them. This sagging will result in an increase in tensile stress in the tie rod as it tends to pull the sheeting. In order to eliminate this condition, one of the following methods may be used:
  - Support the tie rods with light vertical piles at 6m to 10m intervals.
  - Encase the anchor rods in large conduits.

c) Construction Precautions

Removal of soft material and placement of granular fill in the "passive" zone should come first.

6.10 Analysis of Wall for Structural Stability

To assess the material resistance of structural element, analysis of structure is very important to determine the effect of loads on physical structure. To analyse a structure and calculate the forces in the various parts of it with reasonable simplicity and accuracy, it is necessary to represent the structure in a simple manner. The process of replacing an actual structure with a simple system conducive to analysis is called structural idealization. It is a process to transform the proposed engineering structure into a virtual model that represents the physical response of the structure to applied loading. In other words, idealised model is a simplification of the real structure and only considers the characteristics of the components that have the greatest influence on its stability. After idealisation, to carry out structural analysis of a retaining structure, the design engineer needs to adopt one or more models of soil behaviour, and then determine the parameters for the model selected, and for the ground at the site of the proposed structure(s).

The models should be based on sub-soil condition and loadings involved during;

a) Construction stages,
b) Short term or initial service loads and
c) Long term service loads

In general, the simplest soil model that will deliver good predictions should be used for any analysis. The sophistication and rigorous analysis may be necessary or required based on project requirements to capture the response of soil and groundwater complexity and spatial variability etc., if comprehensive geotechnical details are available to idealised model the ground condition properly.

The most frequently used models for soil structure interaction can be classified as follows:

− Rigid plastic models
− Winkler spring models
− Constitutive model for finite element method

6.10.1 Rigid-plastic soil models

The traditional method for design is based in limit equilibrium method. The key consideration of this method is that the wall will rotate about a point near its toe and mobilise active earth pressure and passive resistance in the sides of the wall. In addition, there is the possibility of accounting for pore water pressures, adhesion and friction or even external surcharges or anchors and props. **Figure 6.16** shows a simplified view of the deflection of a rigid cantilever embedded wall moving under lateral loading of soil. As the wall moves in on the soil, the shear stress in soil below excavation level changes and eventually, when the applied shear stress reaches the available shear strength, soil failure will occur. This condition is known as the passive earth pressure state.

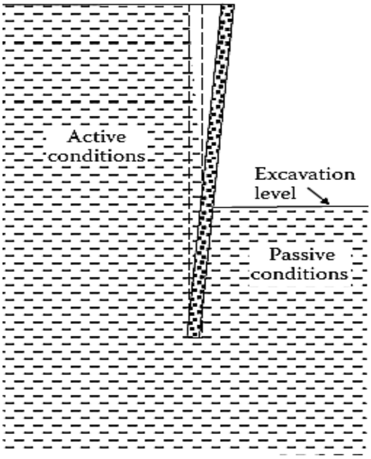


Figure: 6.16 Active and passive states for a propped embedded wall.

A series of assumptions are applied, and horizontal equilibrium and moment equilibrium about the toe are imposed. And finally, the required embedment depth is determined. To ensure the design is conservative an empirical correlation factor (~20%) increases the embedment depth. This allows for the geometric simplifications and analytical assumptions. Analyses based on classical earth pressure theories for active and passive conditions consider that the strength of the ground is fully mobilised regardless of the amount of wall movement, once movement of the wall takes place either away from (active state) or toward (passive state) the soil. As soon as the wall moves toward the soil, it is assumed that, the full soil strength is mobilised to produce passive failure conditions.

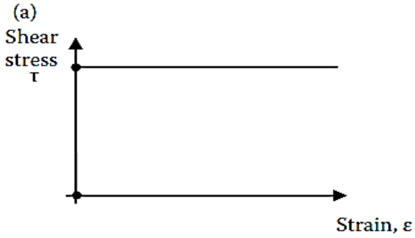


Figure 6.17 (a)
Stress–strain curves for some soil models according to Rigid-plastic behaviour

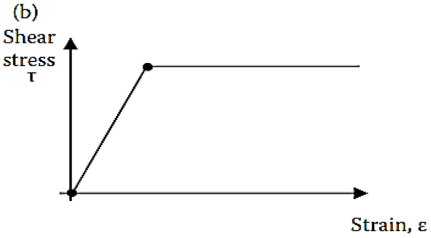


Figure 6.17 (b)
Stress–strain curves for some soil models according to linear elastic plastic behaviour

This is equivalent to considering the soil material to be ‘rigid plastic’, as shown in **Figure 6.17 (a)**. Because the soil is modelled as rigid mass, no strains (or displacements) need to occur before failure is reached. Rigid plastic models require only a definition of failure strength, in two-dimensional (plane strain) conditions. The determination of the bending moment is carried out integrating the distributed forces along the depth of the wall as an elastic beam.

In rigid plastic model, the two typical cases checked are known as free/fixed support. For the first one it is usually assumed zero reaction and moment at both ends of the domain. However, for the fixed case the conditions are the top of the wall are the same but at the bottom the no reaction condition is replaced by the no horizontal displacement condition (rotation permitted). This method is based on the assumption that the piling is driven just deep enough to assure stability, assuming that the maximum possible passive resistance is fully mobilized. The sheet piling is assumed to be inflexible.

6.10.2 Winkler spring models

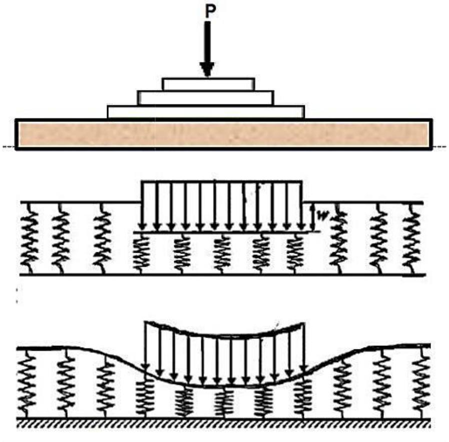
It is much simpler computation tool. Engineers and analysts have made wide use of simpler Computational tools for retaining wall design, with the discrete spring using Winkler spring model. This is based on a conceptual model of soil-structure interaction where the soil support is replaced by a series of discrete, independent springs (Winkler, 1867). Winkler spring models assume that the pressure on a retaining wall or pile is a function of its horizontal displacement toward or away from the soil, up to the point that (active or passive) failure conditions are reached. The wall is modelled as a beam, either using a finite difference or a finite element approximation.

Winkler spring models are a major simplification of wall/soil behaviour. For a retaining wall, differences in overall stress distribution (as well as the stress at a particular section of the wall) will have an effect on the wall displacements. However, the evidence suggests that these relatively simple models can give good estimates of the deformed shape of embedded walls, anchor loads, and shear force and bending moment distributions. It is a sophisticated tool for user interactive analysis, where, load deformation behaviour and limiting deflection case can be judged by designer. Wall deflections and the structural forces in the wall and supports system can be computed, but no information can be obtained on adjacent ground movements.

For virtual modelling of structure in computer and carry out structural analysis, Winkler model requires an estimation of the spring stiffness at different levels of the wall. These are usually based upon measured undrained shear strength (for clays) or density and stress level (for sands and gravels). Computer programs that use this form of analysis will normally give guidance based on experience. It provides reasonable accurate results for 2D analysis of structure without rigorous analysis.

6.11 Analysis of structure using Soil-Structure-Interaction (SSI)

Soil–structure interaction (**SSI**) consists of the interaction between soil (ground) and a structure supported by it. It influences the behaviour of soil, as well as the response of pile under loading. The analysis is highly essential for predicting a more accurate structural behaviour so as to improve the safety of structures under extreme loading conditions. In a laterally loaded pile, the load is resisted by the soil–pile interaction effect, which in turn depends on soil properties, pile material, pile diameter, loading type and bed slope of ground. It is primarily an exchange of mutual stress, whereby the movement of the ground-structure system is influenced by both the type of ground and the type of structure as explained below.



(a) Load applied on ground through a normal rigid beam or plate to induce pressure on ground.

(b) Assumed pressure distribution on ground due to uncoupled spring model.

(c)Actual response of ground due to Soil-structure interaction.

Figure 6.18: Illustration of soil structure interaction on ground

In **Figure-6-18 (a)** ground surface is loaded with a static load through a normal rigid plate. In Winkler foundation model it is assumed that the subgrade soil springs (Winkler foundation) are "uncoupled" discrete springs do not interact with each other. The springs are discrete and independent of each other. Therefore, the soil settlements in one area should not interact with adjacent area as shown in **Figure 6-18 (b)**. But in reality, the stiffness of structure is much greater than the stiffness of soil and for limited settlement case, the overall deformation and moment distribution in structure is not very sensitive to local variation in soil subgrade modulus and actual nature of deformation as shown in **Figure 6-18 (c)** which is expressed as soil-structure interaction. Better understanding on the behaviour of laterally loaded piles or retaining structure can be expressed with SSI model by expressing the properties of soil in terms of subgrade modulus as describe in following sections of this chapter.

6.11.1 Subgrade Modulus of Soil

When a deep retaining sheet pile wall or laterally loaded piles are subjected to lateral loads, the acted loads are transmitted into the ground primarily through horizontal subgrade reaction. The horizontal subgrade reaction is an often misunderstood and misused nomenclature for design of structure; what is the modulus of subgrade reaction? The confusion is added by calling it by many different names in various publications such as modulus of subgrade reaction, subgrade reaction, subgrade modulus, coefficient of subgrade reaction, Winkler foundation (E. Winkler first proposed the subgrade reaction in 1867), Winkler subgrade, “K_h” value, etc.

For laterally loaded piles, Terzaghi defined the horizontal subgrade reaction coefficient (Subgrade Modulus), k_h as:

$$k_h = P / y$$

Where, k_h - Horizontal subgrade reaction coefficient,

P - Load transferred from the pile to the surrounding soil per unit depth,

y - Lateral deflection of the pile at the point considered.

It is often used for structural analysis of embedded retaining walls and it is an important job for geotechnical engineer is to define the subgrade modulus of underlying soil in contact with structure based on the geotechnical studies for site. The mechanical behaviour of soil is very complex due to its non-linear, anisotropic and heterogeneous properties. Hence, in engineering works, for design of the structure, instead of modelling the subsoil in all its complexity, it can be replaced by a much simpler system called a subgrade reaction model.

The subgrade reaction is not a fundamental soil property. It is a lump constant based on sub-soil properties, both the initial response and the long-term response due to sustained loading.

6.11.2 Subgrade Modulus of Soil Based on Load Bearing Capacity of Soil

The principle behind the modulus of subgrade reaction (ks) lies in the resistance of a soil layer with deflection after imposing load to it, similar to a spring shortening with imposed loads. These values are also accompanied by ultimate bearing capacity or load carrying capacity of foundation. Bearing capacity is the measurement of the soil pressure, a soil can safely bear or bearing capacity is the pressure which soil can withstand before it fails.

To evaluate the bearing capacity of ground or sub-soil system, the following two soil failure criteria prevails:

- Shear failure of soil
- Maximum allowable settlement of foundation

In general, to estimate settlement failure, an allowable settlement value is assumed (normally 25 mm or 1 inch; particularly plate load test to estimate the subgrade constant of ground). When soil settles more than the allowable value, the soil fails. For more convenience, the definition of soil subgrade reactions (Ks) is defined as the pressure per unit settlement. In other words, soil capacity to withstand pressure for a given displacement.

Considering the fact that, bearing capacity is correlated with an allowable settlement, the modulus of subgrade reaction is also defined as the bearing capacity of soil per unit settlement.

On above basis JE Bowles established a semi empirical approach to evaluate the modulus of subgrade reaction which is as given below.

$$\text{In SI unit: } Ks = 40(SF)q_a \ \text{kN/m}^3 \quad (6.1)$$

$$\text{In FPS unit: } Ks = 12(SF)q_a \ \text{k/ft}^3 \quad (6.2)$$

Where, SF = Safety factor and q_a is the allowable bearing capacity.

Where, qa is in kPa. This equation is based on assumption that ultimate soil pressure occurs at a settlement of 0.0254 m. For other values of ΔH = 6, 12, 20 mm etc., the factor 40 can be adjusted to 160, 83, 50 etc. 40 is reasonably conservative but smaller assumed displacement can always be used. The most general form for either horizontal or lateral modulus of subgrade reaction is,

$$k_s = A_s + B_s Z^n \tag{6.3}$$

As = constant for either horizontal or vertical members

Bs = coefficient based on depth variation

Z = depth of interest below ground

n = exponent to give ks the best fit.

We know that ultimate bearing capacity is given by,

$$q_{ult} = c.N_c + \gamma Z N_q + 0.5\gamma B N_\gamma \tag{6.4}$$

Observing that,

$$A_s = C\left(c.N_c S_c + 0.5\gamma B N_\gamma S_\gamma\right) \text{ and } B_s Z^n = C\left(\gamma Z^n N_q\right) \tag{6.5}$$

The suggested value for exponent ‘n’ is on the order of 0.4 to 0.6; for granular soil it should be around 0.5.

The *C* factor is 40 for SI units and 12 for FPS; using the same reasoning that q_{ult} occurs at a 0.0254-m and 1-in. settlement but with no safety factor, since this equation directly gives q_{ult}.

Following formula is generally used for finding spring constants representing soil in the model.

$$K_1 = \frac{BL}{3}\,(2k_{s1} + k_{s2})$$

$$K_n = \frac{BL}{3}\,\left(2k_{sn} + k_{s(n-1)}\right)$$

Any other K,

$$K_i = \frac{BL}{6}\,\left(k_{s(i-1)} + 4k_{si} + k_{s(i+1)}\right)$$

Above approach requires Bearing Capacity Parameters (Nc, Nq, Nγ) which is most widely used parameters in foundation engineering because of its simplicity and ease of use and can be appropriately estimated correlating SPT ‘N’ Values and other sub-soil test results.

The other empirical correlation for soil subgrade modulus is given by Vesic. As per Vesic (1961), modulus of subgrade reaction can be computed using stress-strain modulus Es based on as,

$$k'_s = 0.65\sqrt[12]{\frac{E_s B^4}{E_f I_f}}\frac{E_s}{1-\mu^2} \tag{6.6}$$

Where Es, Ef = modulus of soil and footing respectively, in consistent units B, I_f = footing width and its moment of inertia based on cross section in consistent units one can obtain ks from ks’.
Since the twelfth root of any value multiplied by 0.65 will be close to 1, for practical purposes the ***Vesic’s*** equation reduces to,

$$k_s = \frac{E_s}{B(1-\mu^2)} \tag{6.7}$$

Now, we know that immediate (elastic) settlement,

$$\Delta H = q_0 B \frac{1-\mu^2}{E_s} I_f \tag{6.8}$$

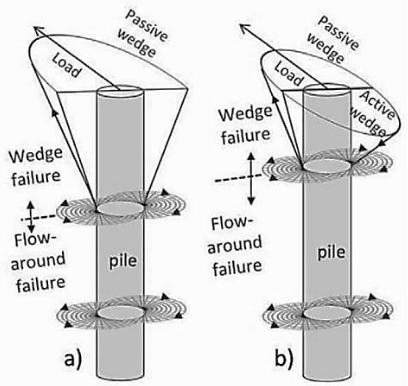
The subgrade reaction modulus, defines the relation between soil pressure and deflection, which is very simple and an efficient parameter that can be used for analysis of a system defining soil structure interaction.

It is most convenient method of analysis for laterally loaded piles or walls involves the theory of subgrade reaction, in which load-deflection relationships for the supporting soil are assumed to be linear functions. However, soil response to lateral loading is highly nonlinear, especially at high stress levels. Lateral soil response is also greatly affected by soil depth and in situ stress conditions. To overcome these difficulties, soil response can be modelled by nonlinear unit load transfer functions, termed *"p-y"* relationships, which are mathematical representations of the soil reaction versus the lateral pile deflection per unit length along the pile.

6.11.3 p-y curve method

The P-y curve is the most extensive method to design the deep pile foundations subjected to lateral loads. It was developed around the seventies [Matlock (1970), Reese et al. (1972), Reese et al. (1974)], and it gives an estimation of the non-linear behaviour of the soil reaction by means of a function that correlates lateral load with deflection at a given depth. Depending on the discretisation level of the pile, the number of non-linear springs can be modelled with a p-y curve. Although this is an empirical method, it has been widely and successfully used in a variety of subsurface conditions. There is a high degree of confidence in the results for most typical applications involving the use of deep foundations that are long enough to be governed by deflection rather than rotation.

There are several *p-y* criteria for deep foundations based on various soil properties and conditions such as (1) soft clay, (2) Stiff clay and (3) Sandy strata above or below the water table. Analytical methods to estimate the ultimate soil resistance near the ground surface were developed based on a wedge type failure theory; whereas, that at some distance below the ground surface was derived based on the flow failure model (Reese *et al*. 2006) as presented in **Figure 6.19**.



for a) gapping and b) no gapping condition

Figure 6.19 Mode of failure of soil along a pile

At a given depth, the prediction of the soil resistance is the most critical factor. For undrained loading, the value of Pu at a depth (*z*) can be estimated using the API model as outlined below;

6.11.4 API model for soft clay

In order to derive the modulus of subgrade for soft clayey soil, the ultimate unit lateral bearing capacity of can be estimated by using the procedure proposed by Matlock (1970). The Matlock procedure has been incorporated into the American Petroleum Institute specification (API RP 2A) for offshore pile foundation design. For static lateral loads, the Matlock procedure calculates the ultimate lateral soil resistance as;

$$Pu = 3c + \gamma X + J\, {}^{cX}\!/_{D} \qquad (6\ 9)$$

In the absence of more definitive criteria, the following is recommended by API. Pu increases from 3*c* to 9*c* as *X* increases from 0 to *XR* according to: (6.-9) and

$$Pu = 9cD \text{ for } X \geq X_R \qquad (6.10)$$

Where, Pu = ultimate resistance, (kPa),

c = undrained shear strength for undisturbed clay soil samples, (kPa),

D = pile diameter, (mm),

γ= effective unit weight of soil, (MN/m^3),

J = dimensionless empirical constant (0.5 for soft clays and 0.25 for medium clay)

X = depth below soil surface, (mm),

X_R = depth below soil surface to bottom of reduced resistance zone in in. (mm). For a condition of constant strength with depth, Equations 6 -9 and 6-10 are solved simultaneously to give:

$$X_R = \frac{6D}{\frac{\gamma D}{c} + J} \qquad (6.11)$$

The area where the strength varies along the depth, Equations 6 -9 and 6-10 may be solved by plotting the two equations, i.e., Pu vs. depth. The point of first intersection of the two equations is taken to be X_R. These empirical relationships may not apply where strength variations are erratic.

Figure 6.19 shows the main characteristics of the static p-y curve in clay has following characteristic.

$$\frac{P}{P_u} = 0.5\left(\frac{y}{y_{50}}\right)^{\frac{1}{3}} \qquad for\ \left(\frac{y}{y_{50}}\right) \le 8;\text{ and} \qquad (6.12)$$

$$\frac{P}{P_u} = 1.0 \qquad for\ \left(\frac{y}{y_{50}}\right) > 8 \qquad (6.13)$$

Where,

p = actual lateral resistance, (kPa),

y = actual lateral deflection, (mm),

$y_{50} = 2.5.\varepsilon_{50}.D$, (mm)

ε_{50} = strain which occurs at one-half the maximum stress on laboratory undrained compression tests of undisturbed soil samples.

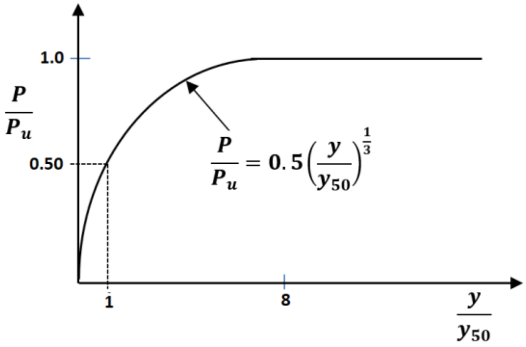


Figure 6.19: p-y curve for Soft Clay

In this relationship, the soil resistance (*p*) is normalized by the ultimate soil resistance (Pu) and is expressed as a function of the lateral deflection (*y*), which is also normalized by the deflection that occurs at 50 percent of the ultimate resistance (y_{50}). The horizontal portion of the *p-y* curve is based on ultimate soil resistance (Pu). Under static loading conditions, the resistance of soft clay increases monotonically as a function of deflection until the ultimate resistance is reached.

For proper analysis of system, obtain the best possible estimate of the variation of undrained shear strength c and submerged unit weight with depth. Also, obtain the value of ε_{50}, the strain corresponding to one-half of the maximum principal stress difference. If no stress-strain curves are available, typical values of ε_{50} are given in table 6-1.

Table 6.1 Typical values for $\boldsymbol{\varepsilon_{50}}$ related to the undrained shear strength

Undrained shear strength Cu (KN /m^2)	$\boldsymbol{\varepsilon_{50}}$
< 12	0.02
12 - 24	0.02
24 - 48	0.01
48 - 96	0.006
96 - 192	0.005
>192	0.004

6.11.5 API model for stiff clay

For static lateral loads the ultimate bearing capacity Pu of stiff clay (c > 1 *Tsf* or 96 kPa) would vary between 8c and 12c, as for soft clay and Pu shall be adopted as lesser of the followings;

$$Pu = 8c + \gamma X + J\,{}^{cX}/_{D} \tag{6.14}$$

$$Pu = 11cD \text{ for } X \geq X_R \tag{6.15}$$

6.11.6 API model for cohesionless soil

For cohesionless soil, the American Petroleum Institute (API) suggested a model to develop p-y curves (API, 1987) based on the methods presented by Reese et al. (1974).

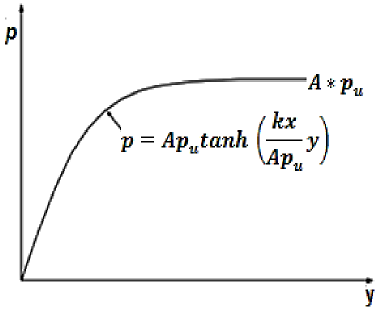


Figure 6.20: p-y curve for API sand

The API model provides simplified calculation procedures and results in an ultimate lateral resistance of similar magnitude to Reese el. API (1974). **Equation** 6-16 and 6-17, presents the API method to calculate lateral soil resistance. Similar to the empirical adjustment factor used by Reese et al (1974), API applies three coefficients to the model as shown in **Figure 6.21**, which are functions of friction angle.

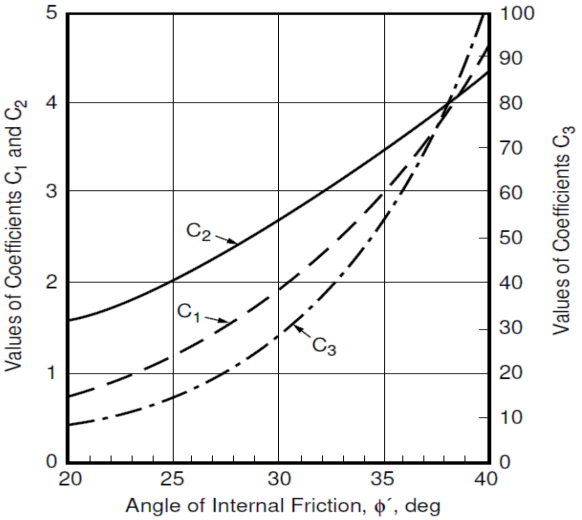


Figure 6.21: Variation of C1, C2, and C3 with the internal angle of friction, from API (1993).

As the ultimate lateral bearing capacity of laterally loaded piles vary from a value at shallow depths (p_{us}) determined by wedge failure Eq. 6-16 to a value at deep depths (p_{ud}) determined by flow failure Eq. 6-17. At a given depth the equation giving the smallest value of p_u should be used as the ultimate bearing capacity.

$$p_{us} = (C_1.H + C_2.D)\,\gamma.H \qquad (6.16)$$

$$p_{ud} = C_3.D.\,\gamma.\,H \qquad (6.17)$$

Where,

p_u = ultimate resistance (force/unit length), lbs/in. (kN/m)

p_{us} = Theoretical Ultimate Soil Resistance due to Wedge Failure

p_{ud} = Theoretical Ultimate Soil Resistance due to Flow Failure

γ = effective soil weight, lb/in.3 (KN/m3),

H = depth, in. (m),

$\emptyset'$ = angle of internal friction of sand, deg.,

C_1, C_2, C_3 = Coefficients from **Figure 6-21** as function of $\emptyset'$,

D = average pile diameter from surface to depth, (m).

k = Coefficient Modulus of Subgrade Reaction from **Table 6-2**

x = Depth along the pile,

The lateral soil resistance-deflection (*p*-*y*) relationships for sand are also non-linear and in the absence of more definitive information and it may be approximated at any specific depth *H*, by the following expression:

$$P = A\,.\,Pu\,.\tanh\left[\frac{k.H}{A.Pu}\,y\right] \qquad (6.18)$$

Where,

A = factor to account for cyclic or static loading condition.

Evaluated by:

A = 0.9 for cyclic loading.

$A = \left(3.0 - 0.8\,\frac{H}{D}\right) \geq 0.9$ for static loading.

p_u = ultimate bearing capacity at depth *H* (kN/m),

k = initial modulus of subgrade reaction (kN/m^3), determine from Table 6-2 as function of angle of internal friction, ϕ´.

y = lateral deflection, inches (m).

H = depth, inches (m)

Table 6-2: Relation of initial modulus of subgrade reaction vs. Relative Density of soil (API)

Angle of Internal Friction ϕ (deg.)	28	29	30	36	40	45
Relative Density (%) of soil	0	20	40	60	80	100
K (KN/m^3) for submerged soil	0	2700	9500	24400	42000	-
K (KN/m^3) for unsubmerged soil	0	2700	12200	43500	75000	-

6.12 Soil models for analysis of retaining structure

In principal, the simplest soil model that will deliver good predictions should be used for any analysis. The sophistication necessary or feasible will be controlled by a number of factors such as quality input data such as quality of geotechnical investigation report and the parameters adopted in design, awareness of stability criteria as how (or where and when) etc. A simple idealised model for anchored wall is shown in **Figure 6-22** below.

In subgrade reaction analyses, it is necessary to input explicitly the stiffness of props or anchors as an equivalent stiffness in the direction normal to the wall.

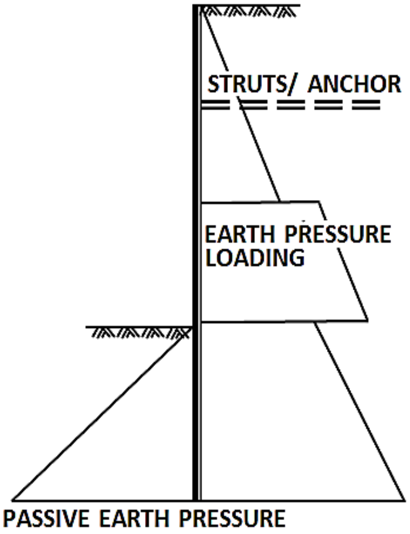

STRUTS/ ANCHOR
EARTH PRESSURE LOADING
PASSIVE EARTH PRESSURE

Figure 6.22(a) Analysis with assumed Soil Loading

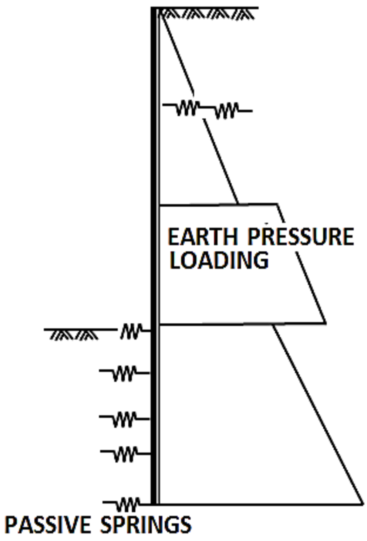

EARTH PRESSURE LOADING
PASSIVE SPRINGS

Figure 6.22(b) Idealised Model of Anchored Retaining Wall

The effective normal stiffness *k* (in kN/m per m run) of a prop should be calculated as follows:

$$k = \frac{A.E}{s.L} \tag{6.19}$$

Where,

A is the prop cross sectional area

E is the Young’s modulus of the prop material

s is the spacing between props

L is the ‘free length’ of the prop, i.e. the distance between the wall and the point at which is the prop is rigidly supported. (For symmetrical walls, the free length is half the length of the prop).

The effective stiffness of the propping system has to be resolved perpendicular to the face of the wall taking account of any inclination of the prop in the horizontal or vertical planes.

6.13 Finite Element Method

The advantage of this method is that they can enable the designer to gain a better understanding of the behaviour of a soil-structure system such as an embedded wall. This is a soil-structure interaction system where the retaining wall derives both loading and support, at least in part, from the ground. The wall transfers load from the retained ground so that it is resisted elsewhere in the soil mass and by the wall and its support system. The manner in which this transfer occurs depends upon the type of wall, the in situ stress state, strength and stiffness of the ground, the wall and the wall's support system and also on the method and sequence by which the wall and its support system are constructed structure and how it interacts with its support system.

The advantages of finite elements methods are (Potts and Zdravkovic, 1999):

-Equilibrium and compatibility requirements satisfied (determination of deflections),

- Utilisation of more advanced constitutive soil models,
- Realistic in situ boundary conditions applied,
- Definition of construction and working states in a single model,
- Interaction between structures can be simulated,
- The analysis predicts behavioural mechanism.

Finite element methods generally provide all the information required from the analysis, take into account all the principles governing the mechanics of the problem and allow to adjust in the design accurately and precisely the key parameters. It is used in practical geotechnical engineering to assess the deformation behaviour of deep excavations, in particular when the influence on existing infrastructure such as buildings or adjacent tunnels has to be evaluated. When using finite elements for retaining wall analysis, there are many factors which are potentially able to affect a desired result. What is required is:

- a knowledge of the relevant factors, and how they can influence the results

  - an awareness of where and when (and why) problems are known to have occurred, and
  - strategies for checking finite element output, to help and ensure that the results are reasonable;

To carry out such analyses requires specialist knowledge and experience in the use of the particular software application. Also, numerical analyses require high quality appropriate input data.

Example 6.3

The undrained shear strength and bulk density of a saturated cohesive soil sample is 20Kpa and 18 KN/cu.m respectively. Using p-y method, compute the subgrade modulus of soil for 1.0m dia. pile upto 20M depth below existing ground level. Assuming the soil is uniform along the depth and ground water table (G.W.T) exists at ground level.

Solution:

Using API model for soft clay;

The ultimate lateral soil $Pu = 3c+\gamma X + J\,{}^{cX}/_{D} \quad for \quad 0 \leq X \leq X_R$

$$= 9cD \text{ for } X \geq X_R$$

$$P = P_u . 0.5 \left(\frac{y}{y_{50}}\right)^{\frac{1}{3}}$$

Subgrade modulus; $\boldsymbol{k = \frac{p}{y_{50}}}$

Here, c = 20 (kPa),

D (pile diameter) = 1.0m,

γ'= 8.0 (KN/m^3),

J = 0.50 for medium clay

ε_{50} = 0.20 (for c = 20 kPa)

$y_{50} = (2.5 * \varepsilon_{50} * D) = 0.05$m

Pu = ultimate resistance, (kPa)

X = depth below soil surface, (m)

p = actual lateral resistance, (kPa)

y = actual lateral deflection

$$X_R = \frac{6D}{\frac{\gamma D}{c}+J} = \frac{6 \text{ x } 1.0}{\frac{8 \text{ x } 1.0}{20}+0.5} = 6.67\text{m}$$

At X =0m; $Pu = \min[(3c+\gamma X + J\, {}^{cX}/_{D}),\ (9cD)]$

$= \min[(3x20 + 8 \text{ x } 0 + 0.5 \text{ x } \frac{20 \text{ x } 0}{1.0}),\ (9 \text{ x } 20 \text{ x } 1.0)]$

= min [60, 180] = 60 Kpa

Lateral deflection (y) at one-half of the maximum stress = y_{50} = 50mm and,

$$P = {}^{P_u}/_{2} = 30 \text{ Kpa}$$

$$k_1 = {}^{p}/_{y_{50}} = \frac{30}{0.05} = 1200\ KN/m^3/m$$

At X =1.0m, $Pu = \min[(3x20 + 8 \text{ x } 1 + 0.5 \text{ x } \frac{20 \text{ x } 1}{1.0}),\ (9 \text{ x } 20 \text{ x } 1.0)]$

= min [78, 180] = 78 Kpa

Similarly, $k_2 = 1560\ KN/m^3/m$

$k_3 = 1920\ KN/m^3/m$

$k_4 = 2280\ KN/m^3/m$

…………………………………………

…………………………………………

At X =7m depth and downward, X = X_R

$Pu = \min[(3x20 + 8 \text{ x } 6.67 + 0.5 \text{ x } \frac{20 \text{ x } 6.67}{1.0}),\ (9 \text{ x } 20 \text{ x } 1.0)]$

= min [180, 180] = 180 Kpa

$k_8 = 3600\ KN/m^3/m$

At various depths, p-y curve of laterally loaded piles can be developed for a set of lateral deflections say; y = 10mm, 25mm, 50mm, 100mm, 200mm, 300mm, 400mm and 500mm as given in Table E-6.1 below. In the absence of actual lateral load test results, the subgrade modulus of pile may be estimated corresponds to y_{50}.

Table E-6.1

X Depth	Pu	Lateral Deflection in Pile								Lateral Resistance in pile								K = p_3/y_3
		y_1	y_2	y_3	y_4	y_5	y_6	y_7	y_8	p_1	p_2	p_3	p_4	p_5	p_6	p_7	p_8	
(m)	(Kpa)	(m)	(m)	(m)	(m)	(m)	(m)	(m)	(m)	(Kpa)	(Kpa)	(Kpa)	(Kpa)	(Kpa)	(Kpa)	(Kpa)	(Kpa)	KN/m^2/m
0	60	0.01	0.025	0.05	0.10	0.20	0.30	0.40	0.50	17.6	23.8	30	37.8	47.6	54.5	60	60	1200
1	78	0.01	0.025	0.05	0.10	0.20	0.30	0.40	0.50	22.8	31.0	39	49.1	61.9	70.8	78	78	1560
2	96	0.01	0.025	0.05	0.10	0.20	0.30	0.40	0.50	28.1	38.1	48	60.5	76.2	87.2	96	96	1920
3	114	0.01	0.025	0.05	0.10	0.20	0.30	0.40	0.50	33.4	45.3	57	71.8	90.4	103.5	114	114	2280
4	132	0.01	0.025	0.05	0.10	0.20	0.30	0.40	0.50	38.6	52.4	66	83.1	104.7	119.9	132	132	2640
5	150	0.01	0.025	0.05	0.10	0.20	0.30	0.40	0.50	43.9	59.5	75	94.5	119.0	136.2	150	150	3000
6	168	0.01	0.025	0.05	0.10	0.20	0.30	0.40	0.50	49.1	66.7	84	105.8	133.3	152.5	168	168	3360
7	180	0.01	0.025	0.05	0.10	0.20	0.30	0.40	0.50	52.7	71.4	90	113.4	142.8	163.4	180	180	3600
8	180	0.01	0.025	0.05	0.10	0.20	0.30	0.40	0.50	52.7	71.4	90	113.4	142.8	163.4	180	180	3600
9	180	0.01	0.025	0.05	0.10	0.20	0.30	0.40	0.50	52.7	71.4	90	113.4	142.8	163.4	180	180	3600
10	180	0.01	0.025	0.05	0.10	0.20	0.30	0.40	0.50	52.7	71.4	90	113.4	142.8	163.4	180	180	3600
11	180	0.01	0.025	0.05	0.10	0.20	0.30	0.40	0.50	52.7	71.4	90	113.4	142.8	163.4	180	180	3600
12	180	0.01	0.025	0.05	0.10	0.20	0.30	0.40	0.50	52.7	71.4	90	113.4	142.8	163.4	180	180	3600
13	180	0.01	0.025	0.05	0.10	0.20	0.30	0.40	0.50	52.7	71.4	90	113.4	142.8	163.4	180	180	3600
14	180	0.01	0.025	0.05	0.10	0.20	0.30	0.40	0.50	52.7	71.4	90	113.4	142.8	163.4	180	180	3600
15	180	0.01	0.025	0.05	0.10	0.20	0.30	0.40	0.50	52.7	71.4	90	113.4	142.8	163.4	180	180	3600
16	180	0.01	0.025	0.05	0.10	0.20	0.30	0.40	0.50	52.7	71.4	90	113.4	142.8	163.4	180	180	3600
17	180	0.01	0.025	0.05	0.10	0.20	0.30	0.40	0.50	52.7	71.4	90	113.4	142.8	163.4	180	180	3600
18	180	0.01	0.025	0.05	0.10	0.20	0.30	0.40	0.50	52.7	71.4	90	113.4	142.8	163.4	180	180	3600
19	180	0.01	0.025	0.05	0.10	0.20	0.30	0.40	0.50	52.7	71.4	90	113.4	142.8	163.4	180	180	3600
20	180	0.01	0.025	0.05	0.10	0.20	0.30	0.40	0.50	52.7	71.4	90	113.4	142.8	163.4	180	180	3600

Example 6.4

The observed undrained shear strength of saturated cohesive soil sample is 100Kpa and bulk density of soil is 19 KN/cu.m. Compute the subgrade modulus of soil for soil-structure-interaction analysis of 1.0m diameter pile applying;

a) p-y method

b) Bowles method to compare the results

The level of G.W.T is at existing ground level.

Solution:

a) Subgrade modulus of soil using p-y method

Using API model for Stiff clay;

The ultimate lateral soil $Pu = 8c + \gamma X + J\, {}^{cX}/_{D}$ $\quad$ *for* $0 \le X \le X_R$

$$= 11cD \qquad \text{for } X \ge X_R$$

$$X_R = \frac{6D}{\frac{\gamma D}{c} + J}$$

$$P = P_u . 0.5 \left(\frac{y}{y_{50}}\right)^{\frac{1}{3}}$$

Here, c = 100 (kPa),

D (pile diameter) = 1.0m,

γ' = 9.0 (KN/m^3),

J = 0.25 for medium clay

ε_{50} = 0.005 (for c = 100 kPa)

$y_{50} = 2.5 * \varepsilon_{50} * D = 0.0125$ m

Pu = ultimate resistance, (kPa)

X = depth below soil surface, (m)

p = actual lateral resistance, (kPa)

y = actual lateral deflection

Subgrade modulus; $\boldsymbol{k} = \frac{\boldsymbol{p}}{\boldsymbol{y_{50}}}$

Subgrade modulus of soil corresponds to y_{50}.at various depth is given below in tabular form.

Table E-6.2

X Depth	Pu	Lateral Deflection in Pile								Lateral Resistance in pile								K = p_1/y_1
		y_1	y_2	y_3	y_4	y_5	y_6	y_7	y_8	p_1	p_2	p_3	p_4	p_5	p_6	p_7	p_8	
(m)	(Kpa)	(m)	(m)	(m)	(m)	(m)	(m)	(m)	(m)	(Kpa)	(Kpa)	(Kpa)	(Kpa)	(Kpa)	(Kpa)	(Kpa)	(Kpa)	KN/m^2/m
0	300	0.0125	0.025	0.05	0.10	0.20	0.30	0.40	0.50	150.0	188.9	238	300.0	300.0	300.0	300	300	12000
1	334	0.0125	0.025	0.05	0.10	0.20	0.30	0.40	0.50	167.0	210.4	265	334.0	334.0	334.0	334	334	13360
2	368	0.0125	0.025	0.05	0.10	0.20	0.30	0.40	0.50	184.0	231.8	292	368.0	368.0	368.0	368	368	14720
3	402	0.0125	0.025	0.05	0.10	0.20	0.30	0.40	0.50	201.0	253.2	319	402.0	402.0	402.0	402	402	16080
4	436	0.0125	0.025	0.05	0.10	0.20	0.30	0.40	0.50	218.0	274.6	346	436.0	436.0	436.0	436	436	17440
5	470	0.0125	0.025	0.05	0.10	0.20	0.30	0.40	0.50	235.0	296.0	373	470.0	470.0	470.0	470	470	18800
6	504	0.0125	0.025	0.05	0.10	0.20	0.30	0.40	0.50	252.0	317.4	400	504.0	504.0	504.0	504	504	20160
7	538	0.0125	0.025	0.05	0.10	0.20	0.30	0.40	0.50	269.0	338.8	427	538.0	538.0	538.0	538	538	21520
8	572	0.0125	0.025	0.05	0.10	0.20	0.30	0.40	0.50	286.0	360.3	454	572.0	572.0	572.0	572	572	22880
9	606	0.0125	0.025	0.05	0.10	0.20	0.30	0.40	0.50	303.0	381.7	481	606.0	606.0	606.0	606	606	24240
10	640	0.0125	0.025	0.05	0.10	0.20	0.30	0.40	0.50	320.0	403.1	508	640.0	640.0	640.0	640	640	25600
11	674	0.0125	0.025	0.05	0.10	0.20	0.30	0.40	0.50	337.0	424.5	535	674.0	674.0	674.0	674	674	26960
12	708	0.0125	0.025	0.05	0.10	0.20	0.30	0.40	0.50	354.0	445.9	562	708.0	708.0	708.0	708	708	28320
13	742	0.0125	0.025	0.05	0.10	0.20	0.30	0.40	0.50	371.0	467.3	589	742.0	742.0	742.0	742	742	29680
14	776	0.0125	0.025	0.05	0.10	0.20	0.30	0.40	0.50	388.0	488.7	616	776.0	776.0	776.0	776	776	31040
15	810	0.0125	0.025	0.05	0.10	0.20	0.30	0.40	0.50	405.0	510.2	643	810.0	810.0	810.0	810	810	32400
16	844	0.0125	0.025	0.05	0.10	0.20	0.30	0.40	0.50	422.0	531.6	670	844.0	844.0	844.0	844	844	33760
17	878	0.0125	0.025	0.05	0.10	0.20	0.30	0.40	0.50	439.0	553.0	697	878.0	878.0	878.0	878	878	35120
18	900	0.0125	0.025	0.05	0.10	0.20	0.30	0.40	0.50	450.0	566.8	714	900.0	900.0	900.0	900	900	36000
19	900	0.0125	0.025	0.05	0.10	0.20	0.30	0.40	0.50	450.0	566.8	714	900.0	900.0	900.0	900	900	36000
20	900	0.0125	0.025	0.05	0.10	0.20	0.30	0.40	0.50	450.0	566.8	714	900.0	900.0	900.0	900	900	36000

b) Subgrade modulus of clayey strata according to Bowles method

The general form for modulus of subgrade reaction by Bowles method is $K_s = A_s + B_s Z^n$

Where, $A_s = C(c.N_c + 0.5\gamma B N_\gamma)$ and $B_s Z^n = C(\gamma Z^n N_q)$

c = 100 (kPa); N_c = 9.0; N_q = 1.0; *η*= 0.5

D (pile diameter) = 1.0m,

γ'= 9.0 (KN/m^3),

C = 40 ; The value of subgrade modulus is given in Table E-6.3

Table E-6.3

Z Depth	Soil Parameters						Subgrade Modulus		
	γ'	c	φ	*Nc*	*Nq*	$N\gamma$	As	Bs	Ks = As + Bs. Zn
(m)	KN/m^3	(Kpa)	Deg.				(Kpa)	(Kpa)	KN/m^2/m
0	9	100	0	9	1	0	36000	0.0	36000.0
1	9	100	0	9	1	0	36000	360.0	36360.0
2	9	100	0	9	1	0	36000	509.1	36509.1
3	9	100	0	9	1	0	36000	623.5	36623.5
4	9	100	0	9	1	0	36000	720.0	36720.0
5	9	100	0	9	1	0	36000	805.0	36805.0
6	9	100	0	9	1	0	36000	881.8	36881.8
7	9	100	0	9	1	0	36000	952.5	36952.5
8	9	100	0	9	1	0	36000	1018.2	37018.2
9	9	100	0	9	1	0	36000	1080.0	37080.0
10	9	100	0	9	1	0	36000	1138.4	37138.4
11	9	100	0	9	1	0	36000	1194.0	37194.0
12	9	100	0	9	1	0	36000	1247.1	37247.1
13	9	100	0	9	1	0	36000	1298.0	37298.0
14	9	100	0	9	1	0	36000	1347.0	37347.0
15	9	100	0	9	1	0	36000	1394.3	37394.3
16	9	100	0	9	1	0	36000	1440.0	37440.0
17	9	100	0	9	1	0	36000	1484.3	37484.3
18	9	100	0	9	1	0	36000	1527.4	37527.4
19	9	100	0	9	1	0	36000	1569.2	37569.2
20	9	100	0	9	1	0	36000	1610.0	37610.0

Example 6.5

An indicator pile of 1.0m diameter is installed upto 20m depth in uniform sandy strata with internal angle of friction as 35 deg. and bulk density as 18 KN/cu.m. Compute the subgrade modulus of soil for soil-structure-interaction analysis of pile applying;

c) p-y method

d) Bowles method to compare the results

Assume, the level of G.W.T is at existing ground level.

Solution:

a) Subgrade modulus of soil according to p-y method

Using API model for cohesionless soil;

The lateral soil resistance-deflection (*p-y*) relationships for sand at any specific depth *H*, can be approximated by the following expression:

$$P = A\,.\,Pu\,.\tanh\left[\frac{k\,.H}{A\,.Pu}\,y\right]$$

Where,

A = 0.9 for cyclic loading.

$A = \left(3.0 - 0.8\,\frac{H}{D}\right) \geq 0.9$ for static loading.

k = initial modulus of subgrade reaction (kN/m^3),

y = lateral deflection (m).

H = depth (m)

p_u = ultimate bearing capacity at depth H (kN/m),

= min (p_{us}, p_{ud})

Where,

$p_{us} = (C_1.H + C_2.D)\,\gamma.H$

$p_{ud} = C_3.D.\,\gamma.\,H$

For,φ =35 deg.,	$C_1 = 3.0$	$C_2 = 3.5$	$C_3 = 35$	$K = 22000$ kN/m^3

D = 1.0m; and $\gamma' = 8$ kN/m^3

At various depths, p-y curve of laterally loaded piles can be developed for a set of lateral deflections say; y = 5mm, 10mm, 12.5mm, 25mm, 50mm, 75mm, 100mm, 150mm and 200mm. The subgrade modulus of pile is estimated corresponds to y_{25} as given below in Table E-6.4.

Table E-6.4

H	A	Pu	$\frac{k.H}{A.P_u}$	y_1	p_1	K = p_1/y_1
(m)		(Kpa)		(m)	(Kpa)	KN/m^2/m
0	3.00	0.0	0.00	0.025	0	0
1	2.20	58.5	170.94	0.025	129	5146
2	1.40	171.0	183.79	0.025	239	9574
3	0.90	337.5	217.28	0.025	304	12150
4	0.90	558.0	175.23	0.025	502	20082
5	0.90	832.5	146.81	0.025	748	29931
6	0.90	1161.0	126.33	0.025	1041	41645
7	0.90	1543.5	110.86	0.025	1378	55133
8	0.90	1980.0	98.77	0.025	1757	70266
9	0.90	2470.5	89.05	0.025	2172	86890
10	0.90	3015.0	81.08	0.025	2621	104837
11	0.90	3613.5	74.41	0.025	3098	123934
12	0.90	4266.0	68.76	0.025	3600	144015
13	0.90	4972.5	63.91	0.025	4123	164925
14	0.90	5733.0	59.69	0.025	4663	186524
15	0.90	6547.5	56.00	0.025	5217	208688
16	0.90	7416.0	52.74	0.025	5783	231309
17	0.90	8338.5	49.84	0.025	6357	254296
18	0.90	8910.0	49.38	0.025	6767	270689
19	0.90	9405.0	49.38	0.025	7143	285727
20	0.90	9900.0	49.38	0.025	7519	300766

b) **Subgrade modulus of soil according to Bowles method**

The general form for modulus of subgrade reaction by Bowles method is $K_s = A_s + B_s Z^n$

Where, $A_s = C(c.N_c + 0.5\gamma B N_\gamma)$ *and* $B_s Z^n = C(\gamma Z^n N_q)$

D (pile diameter) = 1.0m,

γ'= 9.0 (KN/m^3),

n = 0.5

C = 40

For, φ = 35 deg,	N_c = 3.0	N_q = *3.5*	N_γ = 35

The subgrade modulus of soil for pile foundations at various depths are as computed below in Table E-6.5.

Table E-6.5

Depth Z	Soil Parameters						Subgrade Modulus		
	γ′	c	φ	*N*c	*N*q	*N*γ	*As*	*Bs*	Ks = $A_s + B_s . Z^n$
(m)	KN/m3	(Kpa)	deg.				(Kpa)	(Kpa)	KN/m^2/m
0	9	0	35	46.1	50	48	8640	0.0	8640.0
1	9	0	35	46.1	50	48	8640	18000.0	26640.0
2	9	0	35	46.1	50	48	8640	25455.8	34095.8
3	9	0	35	46.1	50	48	8640	31176.9	39816.9
4	9	0	35	46.1	50	48	8640	36000.0	44640.0
5	9	0	35	46.1	50	48	8640	40249.2	48889.2

6	9	0	35	46.1	50	48	8640	44090.8	52730.8
7	9	0	35	46.1	50	48	8640	47623.5	56263.5
8	9	0	35	46.1	50	48	8640	50911.7	59551.7
9	9	0	35	46.1	50	48	8640	54000.0	62640.0
10	9	0	35	46.1	50	48	8640	56921.0	65561.0
11	9	0	35	46.1	50	48	8640	59699.2	68339.2
12	9	0	35	46.1	50	48	8640	62353.8	70993.8
13	9	0	35	46.1	50	48	8640	64899.9	73539.9
14	9	0	35	46.1	50	48	8640	67349.8	75989.8
15	9	0	35	46.1	50	48	8640	69713.7	78353.7
16	9	0	35	46.1	50	48	8640	72000.0	80640.0
17	9	0	35	46.1	50	48	8640	74215.9	82855.9
18	9	0	35	46.1	50	48	8640	76367.5	85007.5
19	9	0	35	46.1	50	48	8640	78460.2	87100.2
20	9	0	35	46.1	50	48	8640	80498.4	89138.4

Chapter 7

Design of Anchors

> *The first rule of sustainability is to align with natural forces, or at least not try to defy them.*
>
> \- *Paul Hawken*

7.1 Introduction

The digging or an excavation in ground causes stress change and variety of stress distribution in the ground particularly around the excavation or dredging area. These stress change cause to lateral deformation and loosening of soil behind the retaining walls, which exerts lateral earth pressure on wall as depicted in **Figure 7.1(a)**.

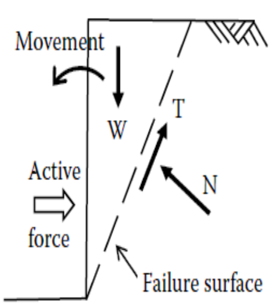


Figure 7.1(a) Mobilisation of soil during Excavation of Ground

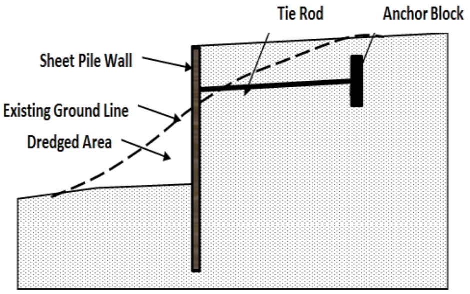
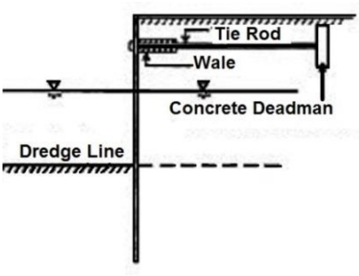


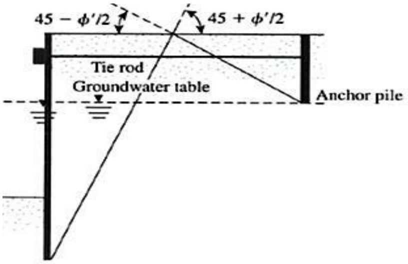
Figure 7.1(b) Anchorage arrangement to control deflection in wall

Sheet piles are flexible structure, if cantilever height of pile is large, the deflection at the top of wall is very large and excessive settlement happens in soil just behind the wall, which may not meet the serviceability requirement. To control the deformation in wall, a lateral support is provided at wall head as shown in **Figure 7.1(b),** known as anchorage. Anchorage is a general expression used to describe the anchoring system at the back of retaining wall. It is generally installed to resist the outwardly directed loads imposed on the structure and transmitted back to a stable earth behind the wall. Anchorage system is an integral part of retaining structure, which not only control the stability of structure, but also add economy in design. It transforms the cantilever embedded pile/beam into a propped cantilever pile or multi-span continuous pile. In consequence of that, it causes to distribute the maximum moment in the pile to reduce structural section and finally the weight of pile. Also due to multiple prop action, limited embedment of pile is required into the ground. Furthermore, a suitable anchor selection may offer a very efficient and economical retaining system.

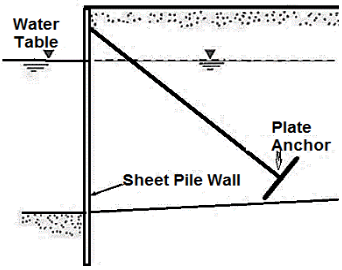
For an anchored retaining wall, the applicable anchors can be classified in two groups as (a) structural anchors and (b) ground anchors as described below. **Figure 7-3** (a, b, c & d) shows the arrangement of structural anchors and **Figure 7-3** (f & g) the ground anchors.


Tie Rod
Wale
Concrete Deadman
Dredge Line

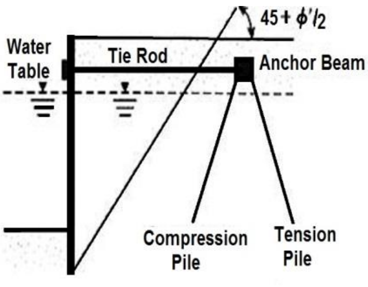
(a) Wall with Deadman Anchor Block


45 − ϕ'/2
45 + ϕ'/2
Tie rod
Groundwater table
Anchor pile

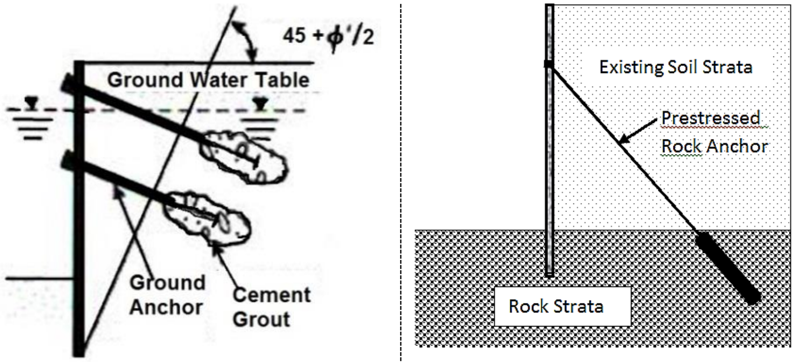
(b) Wall with Cantilever Anchor Pile


Water Table
Plate Anchor
Sheet Pile Wall

(c) Wall with Plate Anchor


45+ ϕ'/2
Water Table
Tie Rod
Anchor Beam
Compression Pile
Tension Pile

(d) Wall with isolated Pile Anchor



(e) Wall with Soil Anchor

(f) Wall with Rock Anchor

Figure 7.2 Various Anchorage Systems for Anchored Wall

7.2 Structural Anchors

Structural anchors may consist of buried concrete blocks, plates, wall or pile elements, which are located at a sufficient distance behind the wall to develop passive resistance.

To support the wall in position, wall needs to be connected with structural anchors through a suitable tie-rods or beams. These types of anchors transmit tensile forces into the ground. Its load carrying capacity is generated as resisting reaction mobilized by passive resistance of ground. The various structural anchors behave differently according to structural arrangement but serve a common purpose. The brief description of anchors commonly applicable for deep retaining walls is given below.

7.2.1 Deadman Anchors

As shown in **Figure 7-2 (a)**; deadmen are short concrete blocks or continuous concrete beams deriving their resistance from passive earth pressure. These types of anchors are suitable when it can be installed below the level of the original ground surface. Tie rods with deadman are typically limited to situations requiring a single level of anchors to secured the wall in position with little or no pre-stressing force. The advantages and disadvantages for use of deadman anchors are given below.

a) Advantages

- Suitable and economical for low height retaining wall.
- Simple construction.

b) Disadvantages

- Not suitable for ground susceptible to settlement.
- Required extra width and excavation work behind the wall for installation. It is not suitable, if area behind the wall is limited.

7.2.2 Cantilever Anchor Wall

Refer, **Figure 7-2 (b)**, cantilever anchored sheet pile walls are non-gravity cantilevered walls. It consists of continuous sheet-piles, either driven or drilled to depths below the finished excavation grade. The wall derives their supports and lateral resistance by virtue of passive pressure of soil below the finished grade in the same manner as deadman. It can be driven from existing ground level to any depth to derive the required passive resistance, provided ground stability conditions are satisfactory. The advantages and disadvantages for use of anchor wall are given below.

a) Advantages

- Suitable for comparatively deep retaining wall.
- Construction methodology is simple and easy to install, if natural finished ground is available.

b) Disadvantages

- Ground shall be stable and settlement in grade is not permitted.
- Extra space behind the wall is required to accommodate the anchor wall and tie-rods.
- Large excavation and backfilling is required for installation of tie-rods.

7.2.3 Plate Anchors

Plate anchors are buried anchors to transmit tensile forces into the ground. Plate anchor assembly normally consists of a buried steel plate or RCC block connected with steel joist to transmit tensile forces into the ground as shown in **Figure 7-2(c)**. Installation of plate anchors requires deep excavation and backfilling; however, it is most viable for site where, deep backfilling is proposed on stable ground. It is applicable for anchored retaining wall; where deep backfilling is required behind the wall, and dead man or tieback construction is not very adaptive. The advantages and disadvantages for use of plate anchors are given below.

a) Advantages

- If deep backfilling is required behind the retaining wall, it is most desirable alternative, provided ground should be stable after placing of backfill.
- Construction methodology is simple.

b) Disadvantages

- Requires special design and fabrication detail. Customised details for use (like tie-rods and pre-stressing strands etc.) are not available in market.

7.2.4 Anchor piles

If any loose granular fill or soft cohesive soils strata exists up to a considerable depth behind the proposed retaining wall, the anchorage system should be supported by group of piles founded in firm strata. **Figure 7.4 (d)** shows an anchor block supported by A-frame piles. The anchor pull force exerted by the tie rod tends to induce compression in front pile and tension in rear pile. This type of anchor system is employed where firm soil is at great depth.

Batter piles are used to resist lateral as well as vertical loads. These are installed at an angle (batter) that typically varies from 1H:6V to 1H:3V. Batter piles resist lateral loads primarily through the horizontal component of the axial load in the pile as determined by the angle of the batter, whereas vertical piles resist lateral loads primarily through structural shear and bending resistance.

The advantages and disadvantages for use of anchor piles are given below.

a) Advantages

- Due to presence of loose or soft compressible strata up to considerable depth, area where anchor wall or deadman cannot be installed, it provides a suitable solution.
- Anchor pile can be used to support the RCC deck platform. Integrating the relieving platform with anchor piles provide safe and economical alternative for construction of waterfront structures.
- Settlement effect of soil can be accommodated with proper design.

b) Disadvantages

- Installation of raker piles required specialised contractors.
- Cost of construction is comparatively high, in comparison to other anchor system.

7.2.5 Ground Anchors or Tie-Back

Ground anchor is a structural element installed in soil that is used to transmit an applied tensile load into the ground. Grouted ground anchors are also referred to as “tiebacks”. It consists of a high tensile steel cable or bar, called the tendon. One end of tendon is held securely in the soil by a mass of cement grout or grouted soil; the other end of it is anchored against retaining wall to be supported, as shown in Fig (7.2 e & f). The main application of ground anchors is to support the retaining walls against lateral pressures and floors against uplift.

There are two types of ground anchors: (1) soil anchors, and (2) rock anchors. Ground anchors transfer load from tendon to grout then from grout to soil or rock. Each consists of a stressing tendon (rod or cable) connecting a fixed anchorage (within the soil or rock mass) to a surface anchor or head. The advantages and disadvantages of ground anchor applications are given below.

a) Advantages

- Pre-stressed ground anchors are “pre-tested” anchors. By pre-stressing each tendon, it is assured that it will hold its design load prior to final construction.
- Fatigue failure is minimized since the service load must exceed the pre-stressed load of the tendon to cause additional steel elongation. Therefore, the periodic stretching and relaxing that causes fatigue failure is eliminated.
- Driven ground anchors can be used in a variety of soil conditions.
- Maximised working spaces can be created on rear side of the wall.
- For braced and mechanised excavation at site, excavation plan can be executed neatly without using props or struts, inside the cofferdam.

b) Disadvantages

- Load carrying capacity of ground anchor is highly dependent on state of workmanship during execution and it is necessary to use specified equipment, experienced professional engineers.
- It is difficult to apply anchors in weak soil and it needs to be installed at greater depth.
- Anchor execution would affect the land of surrounding construction works, which must be accepted by their owners.

7.3 Selection of Anchors

The selection of anchors for a retaining wall is governed by physical and geotechnical conditions of existing ground at site, nature of force, availability of space for construction etc. Factors which must be considered before selection of anchors are;

a) Topography and ground formation at site

b) Physical and mechanical properties of underlying sub soil strata

c) Free space available behind the wall excavation, grilling and construction

d) Ground water condition

e) Sequence of construction and

f) Permissible deformation and serviceability requirement of structure, which shall be discussed in successive sections of this chapter.

7.4 Anchorage System with Tie-Rods

The sheet pile wall is a flexible retaining structure and outward force at top of wall is hold by tie rods and finally tension force is transferred to anchorage system.

The structural arrangement of anchored wall with tie-rods connection is shown in **Figure 7-3**.

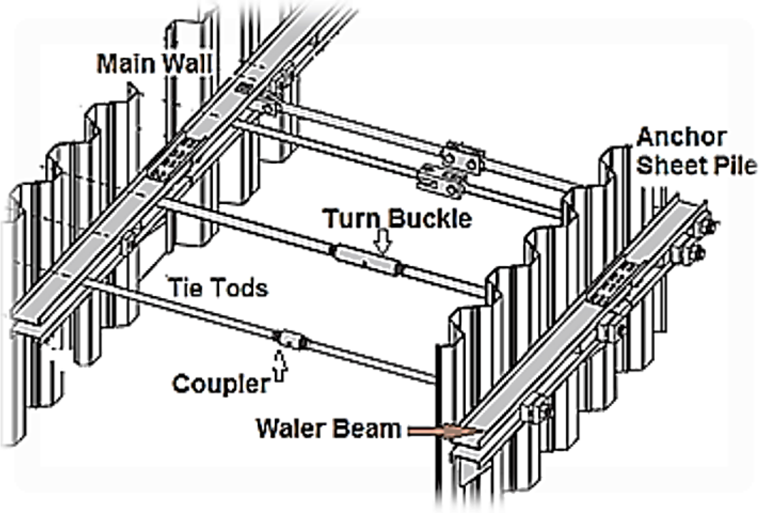


Figure 7.3 Arrangement of Tie-Rod Anchors for sheet Pile wall

The complete anchorage system consists of followings;

i. A series of tie rods placed at certain interval,
ii. Waler beam,
iii. Deadman or anchor wall,
iv. Connectors and articulations

The details of tie-rod arrangement are given below.

7.4.1 Tie Rods

Tie rods counter the pulling forces of wall and serve as tension member. These are generally round steel bar with forged or threaded ends that allow connections to be made to the structure and the deadman. Articulations are provided for adjustment in length of anchors. The tensile resistance of a tie bar depends upon the cross-sectional area of the bar as well as the cross-sectional area of the thread or connecting part to the structure.

On a plain steel round bar threads can be produced by cold rolling, and the minimum tensile area of solid bar will occur in the threaded portion of the bar (see **Figure 7-4(a)** - Thread direct rolled onto bar).

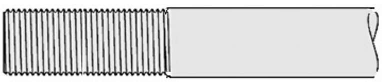


Figure 7.4(a) Thread Direct Rolled **onto bar**

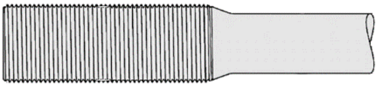


Figure 7.4(b) Upset Forged Thread

Alternatively, threads may be manufactured on upset forged ends which involves forging the end of the parent bar to create a larger diameter over the threaded portion, the minimum tensile area is now in the shaft of the anchor (see **Figure 7-4(b)** - Upset forged thread)

Tie rods are generally threaded at ends that allow connections to be made to the structure and the anchorage. Articulations are provided for adjustment in length of anchors.

7.4.2 Tie Rods Configurations and Connections

The main function of tie rods is to counter the pulling forces of wall and to act as a connector between the wall and the anchorage arrangement. Articulations are provided for adjustment in length of anchors. Tie rod assembly normally comprise of a series of lengths of round steel bar (depending on the maximum length available) coupled together with suitable threaded connections. Connections to the front wall can be made with simple bearing plates and nuts or articulated joints. Several standard tie bar fittings are used to connect multiple tie bars to make longer lengths. Special fittings allow both connection of tie bars and articulation.

Couplers and turnbuckles are used to connect tie bars to make longer lengths. A turnbuckle can be used for length adjustment. The details of Tie Rod assembly are given below. Following type of connectors are generally used which shall be planned as per requirement at site.

7.4.3 Couplers

Couplers are mainly used to connect individual tie rods together, and provide no length adjustment.

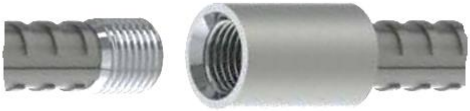


Figure 7-5 Couplers for Tie-Rods

7.4.4 Turnbuckles

The main purpose of turn-buckles is to adjust the length of the total tie rod assembly by up to +/- 50mm. Turnbuckles are right hand-left hand threads connector, which can be installed anywhere on the tie rod, either close to the waling beams, sheet piles or anchor walls, or at mid distance from the walls.

7.4.5 Articulated Joint

Articulation joint are introduced in tie-rods to allow a rotation in vertical direction. It acts like a hinge and minimize/ or reduces the bending stress in tie rods. It may provide rotation at joint up to 90°.

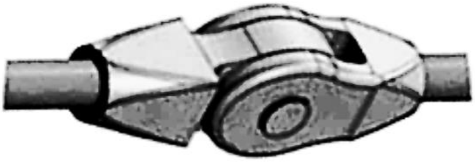


Figure 7.6 Articulated joints with turnbuckles for Tie-Rods

All connecting components to an anchor should be designed to exceed the tensile resistance (Ft) of the bar, hence the bar shaft, or threaded part of bar determines the overall tensile resistance of the anchor. Where the axis of an anchor is not perpendicular to the structure, or sag in tie-rod is expected due to settlement, articulated joints are used. In practice most sheet piled walls are constructed in difficult and less than ideal environments so tie bar connections to walls with some degree of articulation are recommended.

Plates are required to transmit the load imposed on sheet piling to the anchors and from the anchors to the anchorages which may be constructed using sheet piles, tubular piles, and insitu cast concrete or individual concrete blocks. Washer plates are used when the anchors are connected within the plans of sheet piles and bearing plates when the load is transmitted through wailings. When the load is taken to a concrete wall or block, anchorage plates distribute the load to the concrete. The waling loads are transmitted to the anchorages by means of anchor bolts which also require bearing plates and washers of sufficient size to provide adequate bearing on to the sheet piling, wailings etc.

7.5 Design Considerations for Tie rods

For designing of tie rods, following factors should be considered:

a) Tensile resistance of anchor bar
b) Elongation of tie bar under load
c) Settlement of the fill and the effect upon the tie bar and choice of connection design.
d) Corrosion protection of the tie bar

7.5.1 Tensile Resistance

The tensile resistance of a tie bar depends upon the cross-sectional area of the bar and the cross-sectional area of the thread or connecting part to the structure. BS EN1993-5 specifies the tensile resistance of tie-bars shall be adopted as the lesser of the tensile resistance of the thread or the tensile resistance of the shaft, at any time during the life of the structure.

For shaft area (As), the tensile resistance (Fs) of anchor shall be
= **0.6 x fy x As**.

For threaded area (At), the tensile resistance (Ft) of anchor shall be
= **Kt x 0.6 x fy x At**.

The effect of the K_t reduction factor applied on the tensile resistance of the threads to allow for the possibility of bending when settlement occurs in the fill.

To allow bending and tension in the thread of anchor, Kt is recommended as 0.6 at the connections. If structural detailing eliminates bending at connection, Kt shall be adopted as 0.90. To economise the design of tie-rods, use of upset bars allows the cross-sectional area of the thread to be greater than the shaft. Particularly in seismic areas, it is recommended that failure should occur in the shaft first, and not in the threads, as this will allow greater and more uniform deformation to occur before failure.

7.5.2 Grades of Steel for Tie-rods

The most common steel grades of material available for tie-rods include 355, 460, 500 and 700 grades (which typically have yield values, f_y, of 355, 460, 500 & 700 N/mm^2 respectively). The choice of steel grade depends on a number of factors, whilst the higher strength steel will always produce the anchor of lightest weight. However, this may not be suitable for stiffness requirements, onsite welding or lead-times. It is recommended that high strength steel grades (f_y more than 500 N/mm^2) shall be used with caution.

7.5.3 Elongation in Tie Rods

Elongation in tie rods under the design load should be checked, this should be based on the shaft diameter of the tie bar. Use of higher grade of steel bars economise the tie-rod section, but reduced cross-sectional area increases the elongation in bars, which shall be checked with permissible deflection in wall. The increased stiffness of the threaded area (if larger) can be neglected as tie bars are typically sufficiently long for the effect to be negligible.

7.5.4 Settlement Effect

When detailing tie bars, it is important to consider the effect of sag of the anchor and forced deflection due to settlement of fill or for any differential settlement where ground conditions change (e.g. from existing ground to new fill). Bending and shear stresses can be induced at a fixed connection when a tie rod is displaced or when fill settles causing the tie bar to deflect (see **Figure 7-7**). The effect of fill settlement in tie bar should be accounted in the design as described in section 7.4.1. The Bending at the thread can be minimised by introducing the articulations. Where large settlements are expected further articulation may be required between the front and anchor walls.

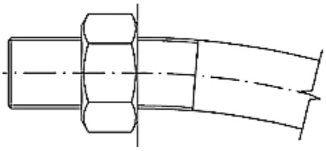


Figure 7.7 Bending Stresses in Thread by Settlement or Misalignment

Settlement may occur in a fill (even if mechanically compacted) and it is good practice to minimise the effect upon the tie bar by using articulated joints at appropriate locations and allowing the bar to rotate with settlement. Settlement ducts are sometimes used as a means of reducing the effect of settlement on the tie bar. Tie bars are placed inside large (typically 200-350mm) plastic tubes, the bar lies on the bottom of the tube and as fill settles the tube settles leaving the bar suspended in the tube. The bar will still deflect due to its own self weight and articulation should still be supplied to the wall connection.

7.5.5 Corrosion Protection

In aggressive environment, steel tie-rods are susceptible to corrosion attack and it affects the design life of tie bars. Therefore, it is important to consider the corrosion protection for the tie bars at design stage. The most common and robust method is to allow sacrificial thickness for corrosion loss in the section of tie bar i.e. the diameter of tie bar is to allow for anticipated corrosion. In this situation, consideration should be given to the probable corrosion rates and consequential loss of tie bar section, in the thread, shaft and fittings depending on their position in the structure. In addition to above, to ensure the safety of structure, other corrosion protection method, such as painting, galvanising or wrapping of tape on tie-rod should be applied. The most commonly used method is to wrap the anchor to give an appropriate level of corrosion protection. Often the anchor shaft is wrapped in factory conditions and shipped to site but connections cannot be wrapped until installed on site.

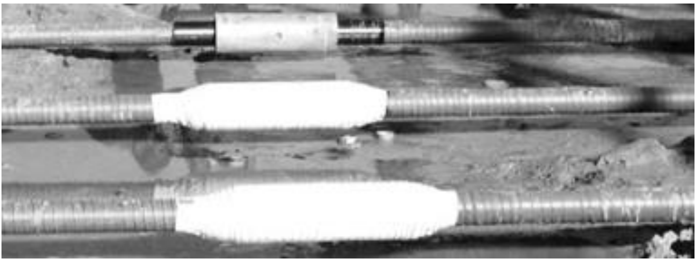


Figure 7.8 Wrapping of connection on site

The vulnerable anchor head should be fully protected as shown in **Figure 7-8**. Any breaks in the wrapping system could lead to aggressive pitting corrosion and premature failure of the anchor. Further protection should also be given by judicious detailing of tie bar so that connections at wall must be away from the most aggressive regions of corrosion.

7.5.6 Wales

The tensile force from tie rod is transferred to the anchor wall by a flexural member known as a wale or waler. The waler is designed to distribute the anchor load evenly across the sheet pile wall and provide a point of attachment for the threaded bar. The waler distributes the load from the main wall to an anchor system. It normally consists of two spaced structural steel channels placed with their webs back to back in the horizontal position. **Figure 7.9** shows common arrangements of wales and tie rods located at anchor wall.

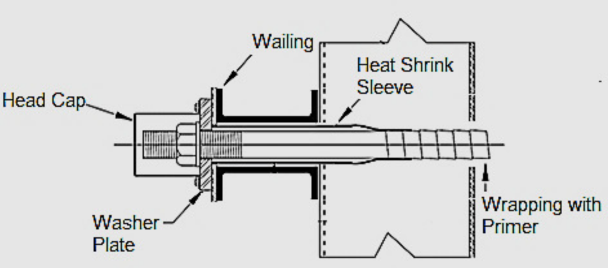


Figure 7.9 Arrangement of Waler Beam

The channels are spaced with a sufficient distance between their webs to clear the upset end of the tie rods. Battened plates at certain spacing are generally used to fabricate the wales and maintain the required spacing between channels. If wales are constructed on the inside face of the sheet piling, every section of sheet piling should be bolted to the wale to transfer the reaction of the piling. The best location for the wales is outer face of the wall, where the piling bears against the wales. Wherever, the excavation work needs to be limited within the wall face, they are generally placed inside the wall to avoid outside excavation.

7.5.7 Anchor Block or Deadmen Anchor

Marine bulkheads and retaining structures are generally supported by a single level of anchor and tie-rod connecting the wall and anchor block or "deadman" located at some distance behind the wall. Such anchors develop their resistance by virtue of their deadweight and the passive resistance of the ground between the anchor and the wall as shown in **Figure 7-10.** The mechanism of load balancing activity of an anchor associated with tie-bar is shown in **Figure7-10**.

Such anchors can be constructed of mass concrete, sheet-piles, a parallel wall structure, or piles (driven or drilled). The load capacity of a "deadman" anchor is highly dependent on its placement relative to the wall.

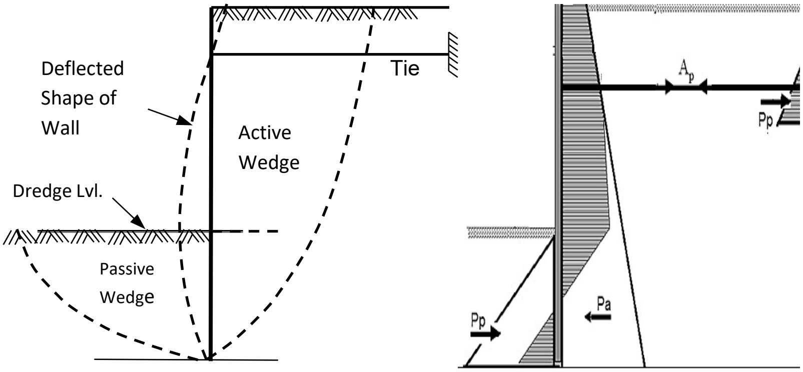


Figure 7.10 Balancing of Lateral Loads in Anchored Retaining Wall

7.6 Location of Anchorage

In order for an anchorage system to be effective it must be located outside the potential active failure zone developed behind a sheet pile wall. Its capacity is also impaired if it is located in unstable ground or if the active failure zone prevents the development of full passive resistance of the system. **Figure 7.11** shows several installations that will not provide the full anchorage capacity required because of failure to recognize the above considerations.

- Anchor block A is located inside active wedge and offers no resistance.
- Anchor block B, resistance is reduced due to overlap of the active wedge (wall) and the passive wedge (anchor).
- Anchor block C develops full capacity but increases pressure on wall.
- Anchor block D develops full capacity and has no effect on bulkhead.

Anchor blocks should be placed against firm natural soil and should not be allowed to settle. A safety factor of 2 is recommended for all anchors and anchor blocks.

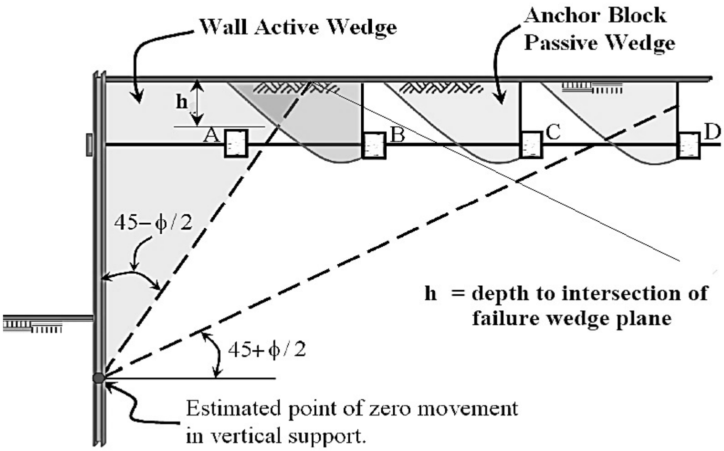

Wall Active Wedge
Anchor Block Passive Wedge
h
A
B
C
D
45−ϕ/2
45+ϕ/2
h = depth to intersection of failure wedge plane
Estimated point of zero movement in vertical support.

Figure 7.11 Shows the Effect of Anchorage Location on the Resistance Developed

7.7 Design of Anchor Block or Deadman

Case A – Anchor block extends to the ground surface

The forces acting on an anchor are near the ground surface is shown in **Figure 7.12**. The capacity of an anchor block also depends on whether it is continuous or isolated. An anchor block is considered continuous when its length greatly exceeds it height.

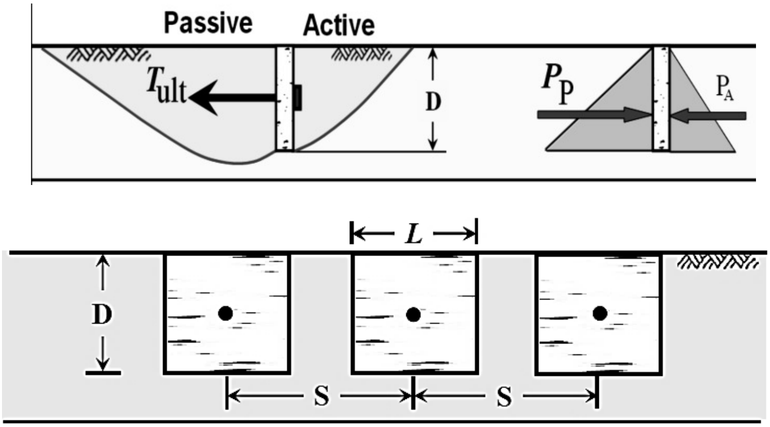

Passive
Active
T_{ult}
D
P_P
P_A
L
D
S
S

Figure 7.12 Anchor block in cohesionless soil near ground surface

The conventional earth pressure theories using two-dimensional conditions corresponding to a long wall can be used to calculate the resistance force against the anchor block movement. The ultimate capacity, T_{ult}, of a deadman anchor equals the total passive lateral earth pressure resistance minus the total active lateral earth pressure minus any lateral earth pressure due to surcharge loads acting behind the anchor, see **Figure 7.12**. The basic equation for a continuous anchor block is shown below:

In case of isolated and short anchor blocks if, d < D/2 (as shown in **Figure 7-13**), it may be assumed that the anchor extends to the finished grade and the ultimate capacity of the anchor is,

$$T_{ult} = (Pp - Pa)\ L \tag{7.1}$$

Where,

T_{ult} = ultimate capacity of the concrete anchor

Pa = total lateral active earth pressure acting on height, *D*, per unit length of anchor

$$= K_a\ \gamma\ \frac{D^2}{2}$$

Pp = total lateral passive earth pressure acting on height, *D*, per unit length of anchor

$$= K_p\ \gamma\ \frac{D^2}{2}$$

D = virtual height of concrete anchor

L = actual length of concrete anchor

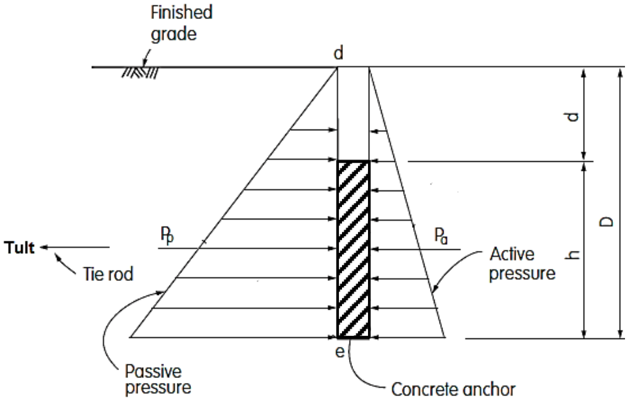

Finished grade
d
d
D
h
Tult
Tie rod
P_p
P_a
Active pressure
e
Passive pressure
Concrete anchor

Figure 7.13 Continuous Deadmen near Ground Surface

For design in cohesive soils, both the immediate and the long-term pressure conditions should be checked to determine the critical case. A safety factor of two (2) against failure is recommended.

7.8 Short Deadman near Ground Surface

Figure 7.14 shows a deadman of length, L, located near the ground surface, subjected to an anchor pull, T. Experiments have indicated that at the time of failure, due to edge effects, the heave of the ground surfaces takes place in an area as shown in **Figure 7.14**. The surface of sliding at both ends is curved.

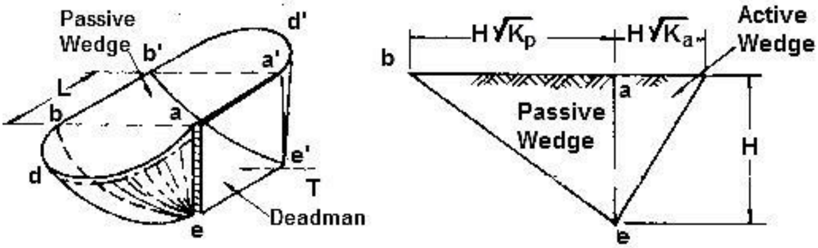

Passive Wedge
d'
b'
a'
L
b
a
e'
d
T
e
Deadman
b
$H\sqrt{K_p}$
$H\sqrt{K_a}$
Active Wedge
a
Passive Wedge
H
e

Figure 7.14 Short Deadman near ground surface

Integration of the resistance along these curved sliding surfaces results in the following expression for the ultimate capacity of short deadmen in granular soils,

$$T_{ult} \leq L\left(P_p - P_a\right) + \frac{1}{3} K_o \gamma \left(\sqrt{K_p} + \sqrt{K_a}\right) D^3 tan\emptyset \qquad (7.2)$$

Where, T_{ult} = ultimate capacity of the deadman,

L = length of the deadman, feet

P_p = total passive and active pressure,

K_o = coefficient of earth pressure at rest.

γ = unit weight of soil,

K_p, K_a = coefficients of passive and active earth pressure.

D = Depth of deadman.

ϕ = angle of internal friction.

For cohesive soils, the second term in the above expression should be replaced by the cohesive resistance, thus

$$T_{ult} \le L\left(P_p - P_a\right) + 2cH^2 \tag{7.3}$$

Where, c = the cohesion of the soil,

Tie rods with deadman are typically limited to situations requiring a single level of and secured to the wall with little or no prestress force.

Example 7.1

For a low height anchored retaining wall, continuous deadman anchor (wall) is proposed well behind the wall in passive zone. The force transferred through the anchor is 215 KN/m. Check the geotechnical stability of RCC deadman [as shown in Figure E-7.1(a)] taking care of following parameters.

Parameters given for design for anchor

Finished ground level	=	*+ 5.0*	*m Lvl.*
Top Level of RCC deadman (wall)	=	*+ 4.0*	*m Lvl.*
Bottom Level (feet) of deadman	=	*+ 1.0*	*m Lvl.*
Total depth of deadman (D)	=	*3.0*	*m*
Thickness of wall (tw)	=	*0.4*	*m*
Base width of wall (B)	=	*2.0*	*m*
Thickness of base (tb)	=	*0.4*	*m*
GWT (adopted for design) (hw)	=	*+ 1.0*	*m Lvl.*
Design Live Load Surcharge on backfill (q)	=	*50.0*	*KN/m^2*

ϕ of backfill & founding material = *32* *deg.*

Unit wt. of soil (γ') = *18.0* *KN/m^3*

Unit wt. of concrete = *24.0* *KN/m^3*

K_0 = *0.47*

Ka = *0.31*

Kp = *3.25*

Solution:

i) Calculation of Forces

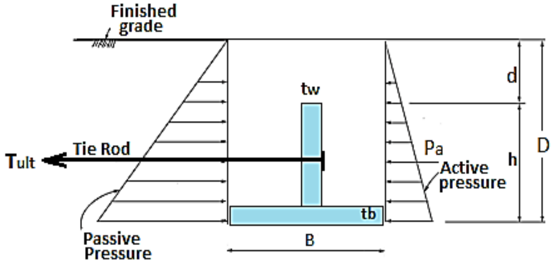

Finished grade
tw
d
D
Tie Rod
Tult
Pa
Active pressure
h
tb
Passive Pressure
B

Figure E:7-1(a)

From above figure E:7-1(a) D = 4.0 m

h = 3.0 m

d = 1.0 m

d < D/2; force will be applicable upto the finished G.L.

$\sigma_a(1) = Ka.\gamma, D$ = 22.12 KN/m^2

Normal Active Earth Pr. P1$= 0.5.\ K_a.\gamma, D^2$ = 44.25 KN /m

Vertical Pr. due to Live Load surcharge = Ka .q = 15.36 KN/m^2

Active Earth Pr. due to Live Load surcharge

Hs = Ka .q. D = 61.45 KN/m

Total active earth Pr. = Pa = P1 + Hs = 105.70 KN/m

Normal Passive Earth Pr. $Pp = 0.5.\ K_p.\gamma, D^2$ = 468.66 KN /m

ii) Calculation of frictional resistance at the base of wall

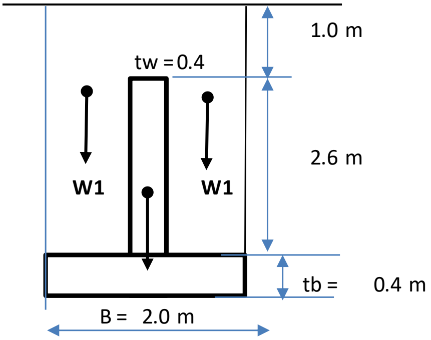


Figure E:7-1(b)

From above figure E:7-1(b);

Weight of concrete deadman structure (W) = 44.16 KN/m

Weight of soil on deadman base (2 * W1) = 96.48 KN/m

Total vertical reaction at base (N = W + 2.W1) = 140.64 KN/m

Coefficient of friction at base $\mu = \tan\emptyset$ = 0.62

Frictional resistance at base $\mu.N$ = 87.88 KN/m

Natural stabilising force on Deadman = Pp + μ.N = 556.54 KN/m

Natural destabilising force on Deadman = Pa = 105.70 KN/m

Net stabilising force = Pp + μ.N - Pa = 450.85 KN/m

Factor of safety against sliding (FS) = 2.0

Safe anchorage capacity Ta = (Pp + μ.N - Pa) /FS = 225.42 KN/m

Ta > Tult (Hence, Safe)

Example 7.2

To anchor the head of a retaining wall, isolated deadman anchor of 2.5m length in the form of strip footing is proposed. The total force transferred through the anchor is 675 KN. Check the adequacy of deadman anchor for following parameters.

Parameters given for design for anchor

Finished ground level = + 5.0 m Lvl.

Top Level of anchor wall = *+ 4.0* *m Lvl.*

Bottom Level of anchor wall = *+ 1.0* *m Lvl.*

Total depth of anchor Wall (D) = *3.0* *m*

Thickness of wall (tw) = *0.4* *m*

Base width of wall (B) = *2.0* *m*

Length of deadman block (L) = *2.5* *m*

Thickness of base (tb) = *0.4* *m*

GWT (adopted for design) (hw) = *+ 1.0* *m Lvl.*

Design Live Load Surcharge on backfill (q) = *50.0* *KN/m*^{2}

ϕ *of Backfill & Founding material* = *32* *deg.*

Unit wt. of soil (γ') = *18.0* *KN/m*^{3}

Unit wt. of concrete = *24.0* *KN/m*^{3}

K_o = *0.47*

Ka = *0.31*

Ka = *3.25*

i) Calculation of Forces

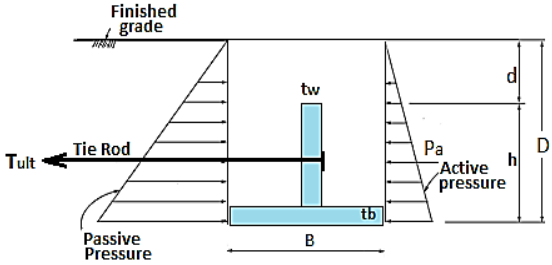

Finished grade
tw
Tie Rod
Tult
Pa
Active pressure
d
h
D
tb
Passive Pressure
B

Figure E:7-2(a)

From above figure E:7-2(a) D = 4.0 m

h = 3.0 m

d = 1.0 m

d < D/2; force will be applicable upto the finished G.L.

$\sigma_a(1) = Ka.\gamma.D$ = 22.12 KN/m^2

Normal Active Earth Pr. P1= $0.5.\ K_a.\gamma, D^2$ = 44.25 KN /m

Vertical Pr. due to Live Load surcharge = Ka .q = 15.36 KN/m^2

Active Earth Pr. due to Live Load surcharge

Hs = Ka .q. D = 61.45 KN/m

Total active earth Pr. = Pa = P1 + Hs = 105.7 KN/m

Normal Passive Earth Pr. $Pp = 0.5.\ K_p.\gamma, D^2$ = 468.66 t /m

ii) Calculation of friction force at the base of wall

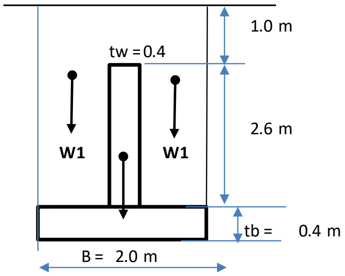


Figure E:7-2(b)

Weight of concrete deadman structure (W) = 44.16 KN/m

Weight of soil on deadman base (2 * W1) = 96.48 KN/m

Total vertical reaction at base (N = W + 2.W1) = 140.64 KN/m

Coefficient of friction at base = $\mu = \tan\emptyset$ = 0.62

Frictional resistance at base $\mu. N$ = 87.88 KN/m

Natural stabilising force on Deadman = Pp + µ.N = 556.54 KN/m

Natural destabilising force on Deadman = Pa = 105.70 KN/m

Net stabilising force = (Pp + µ.N - Pa) .L = 1127.1 KN

Additional force due to wedge action =

1/3 $Ko\ \gamma\ (\sqrt{Kp} + \sqrt{Ka})\ D$^3 $tan\emptyset$ = 266.0 KN

Ultimate sliding resistance = 1393.13 KN

Factor of safety against sliding (FS) = 2.0

Safe anchorage capacity Ta = 696.56 KN

Ta > Tult (Hence, Safe)

7.9 Cantilever Anchor Pile (wall)

Short or medium length steel sheet piles driven in the form of a continuous wall can be used to anchor tie rods. The tie rods are connected with a waling system similar to that for the "parent" wall, and resistance is derived from passive pressure developed as the tie rod pulls against the anchor wall. To provide some stability during installation of the piling and the wales, pairs of the piling should be driven to a greater depth at frequent intervals. The anchor wall is analysed by conventional means considering full passive pressure developed only if the active and passive failure zones do not intersect.

For design, the ultimate capacity, T_{ult}, of an anchor wall is a function of the moment resistance of the total passive lateral earth pressure minus the driving moment of the total active lateral earth pressure and any lateral earth pressure due to surcharge loads acting on the anchor pile embedment, D, and anchor pile effective width, b, see Figure 7-15. If, d < b (width of pile), it may be assumed that the anchor pile extends to the finished grade and that the ultimate capacity of the anchor pile is;

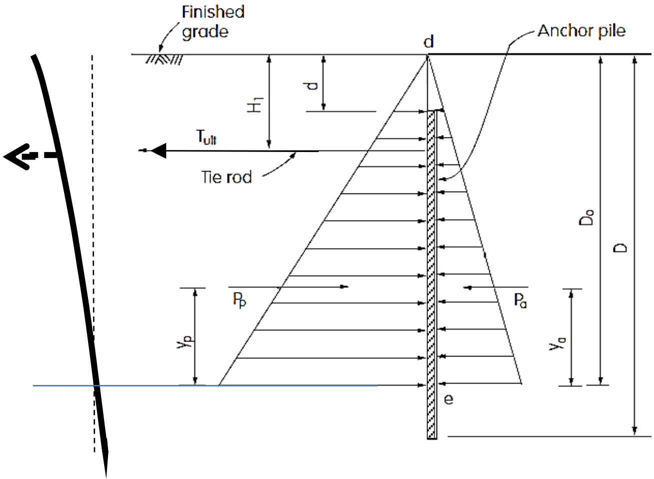


Figure 7.15 Continuous Anchor wall Near Ground Surface

$$T_{ult} = \frac{(P_p{\cdot}y_p - P_a{\cdot}y_a)}{(Do - H_1.)} \qquad (7.4)$$

Where,

T_{ult} = ultimate capacity of the anchor pile

Pp = total lateral passive earth pressure acting on height, *Do*, and effective anchor pile width, *b*

Pa = total lateral active earth pressure acting on height, *Do*, and effective anchor pile width, *b*

yp = vertical distance from the bottom of embedment, *Do*, to the level at which, *Pp*, acts

ya = vertical distance from the bottom of embedment, *Do*, to the level at which, *Pa*, acts

Do = depth of rotation in pile

H_1 = distance from finished grade to level at which anchor pull acts on pile

b = effective width of anchor pile

D = embedment from finished grade to be used for anchor pile

The passive resistance of anchor wall should be estimated based on soil structure interaction analysis accounting actual deformation in wall due to anchor pull as shown in **Figure 7-15**. It is effective upto the depth of wall moving against the soil, which is governed by the stiffness of soil and structural rigidity of wall. It must be noted that laterally loaded piles shift horizontally in response to applied load which results in bending, rotation or translation of pile. In majority of cases, anchor piles are embedded in soil and are non-socketed and lateral capacity of these piles depends to a larger extent on the stiffness of pile and properties of soil. A short rigid (anchor) pile, unrestrained at the top tends to rotate about a point slightly above the bottom tip and passive resistance develops in the portion above the point of rotation.

Here, the goal of designer is to determine deflections, rotations, and moments and shear forces of piles in a selected soil-pile system. Piles deflection and rotation is used to assess the extent of passive resistance of soil mass develop against the piles, whereas moments and shear forces are utilised to verify the internal load resistance of pile structure.

Example 7.3

A 15m deep cantilever anchor of adequate structural strength is installed behind a retaining wall to transfer 850 KN/m force into the passive zone of soil. The head of retaining wall and anchor wall is connected through a series of tie rods placed at 2.5m spacing. After soil structure interaction analysis of anchor wall, depth of fixidity of wall is observed at 13m depth. Calculate the geotechnical capacity of anchor wall considering the following parameters.

Parameters given for design for anchor

Force at anchor level (Tu) = 850.0 KN/m
Finished ground level = + 6.0 m Lvl.
Top Level of anchor wall = + 5.0 m Lvl.
Bottom Level of anchor wall = - 10.0 m Lvl.
Total depth of anchor Wall (D) = 15.0 m
Level of Tie rods for anchorage = + 2.5 m Lvl.
Depth of Fixidity of Wall (Do) = - 8.0 m Lvl.
GWT (adopted for design) (hw) = + 2.0 m Lvl.
Design Live Load Surcharge on backfill (q) = 50.0 KN/m^2
Spacing between Tie-Rods = 2.5 m c/c
ϕ *of Backfill & Founding material = 32 deg.*
Unit wt. of soil (γ') *= 18.0* KN/m^3
K_0 *= 0.47*
Ka = 0.31
Ka = 3.25

i) Calculation of Forces

From figure shown below;

Tu = 850.0 KN
H1 = 3.5 m
D = 16.0 m
Do = 14.0 m
d = 2.0 m

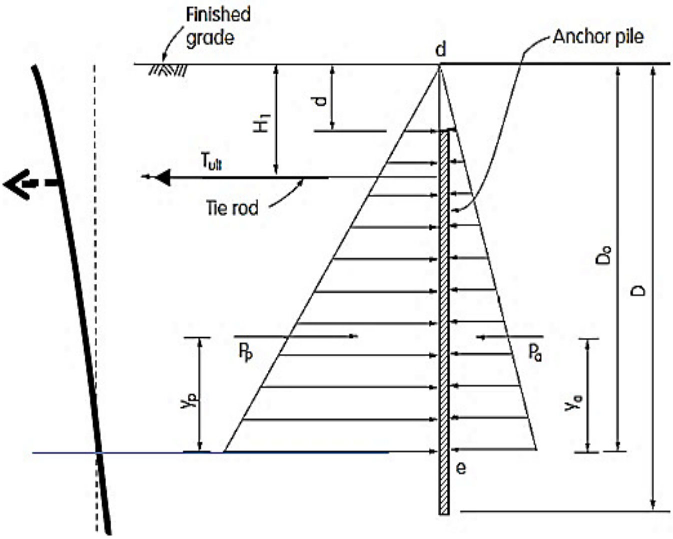


Figure E:7-3

Since, this is a continuous interlocked wall, the effective width of wall (b) is considered as the spacing of Tie-rods D < b; force will be applicable upto the finished G.L.

$\sigma_a(-8) = Ka.\gamma.Do$ P1= $0.5.\ K_a.\gamma.Do^2$ = 77.43 KN/m^2

Normal Active Earth Pr. = 542.0 KN /m

C.G of active earth Pr. From point 'e' (y_a) = 4.67 m

Vertical Pr. due to Live Load surcharge = Ka .q = 15.36 KN/m^2

C.G of pressure due to surcharge from point 'e' (y_s) = 7.00 m

Active Earth Pr. due to Live Load surcharge

Hs = Ka .q. D = 215.08 KN/m

Total active earth Pr. = Pa = P1 + Hs = 757.1 KN/m

Normal Passive Earth Pr. $Pp = 0.5.\ K_p.\gamma.Do^2$ = 5741.1 KN/m

C.G of passive earth Pr. From point 'e' (y_p) = 4.67 m

ii) **Check for Stability of Wall**

Natural stabilising force on anchor wall Pp = 5741.1 KN/m

Total destabilising force on Wall = Pa + Hs + Tu = 1607.1 KN/m

$$Pp > \ Pa + Hs + Tu$$

$$^{P_p}\!/_{P_a + H_s + T_u} = 3.57$$

F.O.S. in wall against horizontal forces = 3.57 (O.K)

$$T_{ult} = \frac{(P_p.y_p - P_a.y_a - H_s.y_s)}{(D_0 - H_1)}$$

$P_p.y_p$ = 26792.0 KN.m

$P_a.y_a$ = 2529.4 KN.m

$H_s.y_s$ = 1505.6 KN.m

T_{ult} = 2167.3 KN/m

$$^{T_{ult}}\!/_{T_u} = 2.55$$

F.O.S. in wall against moment due to forces = 2.55 (O.K)

7.10 Plate Anchors

Retaining structures requires foundation systems to resist horizontal pull-out forces. However, an economical design solution may be achieved through the use of tension members. The pull-out capacity of tension member is limited by the stability of anchor fixing arrangement. For an instance, the tension capacity of tie-rod is governed by the passive resistance of soil developed against the deadman or anchor wall. Plate anchors are modified form of deadman system. Plate anchors assembly normally consists of a buried steel plate or RCC block connected with steel joist to transmit tensile forces into the ground as shown in **Figure 7-16**.

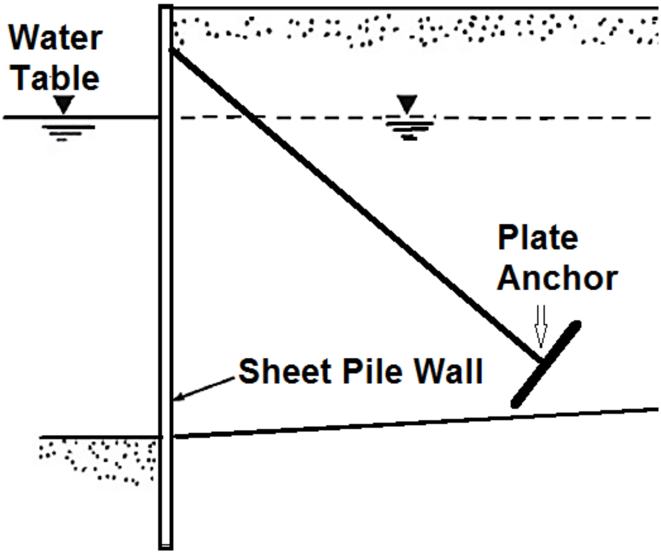


Figure 7.16 Inclined Plate Anchor for Retaining Wall

It is used to transmit tensile forces from a structure to the soil may be circular, square or rectangular in shape. Installation of plate anchors requires deep excavation and backfilling; however, it is most viable for site where, deep backfilling is required on stable ground. It is applicable for anchored retaining wall, where deadman or tieback construction is not very adaptive and deep backfilling is required behind the wall.

Their strength is obtained through the shear strength and dead weight of the surrounding soil. Anchors plates are typically constructed from steel or concrete and for retaining structure; they may be placed horizontally, or at an inclined position depending on the load orientation or type of structure requiring support. It can be installed by excavating the ground to the required depth, placing the anchor, and then backfilling with soil. If plate anchor is located at a shallow depth, the failure surface at ultimate load will extend to the ground surface. However, as the depth of anchor increases, the breakout factor q_u *i*ncreases with embedment ratio *Ha /B.*

*A*t a given embedment ratio, $Ha/B = (Ha/B)_{\text{critical}}$, the magnitude of q_u remains practically constant. The critical embedment ratio $(Ha/B)_{\text{critical}}$ and the maximum breakout factor *qu* are functions of ϕ and *B*/*h.*

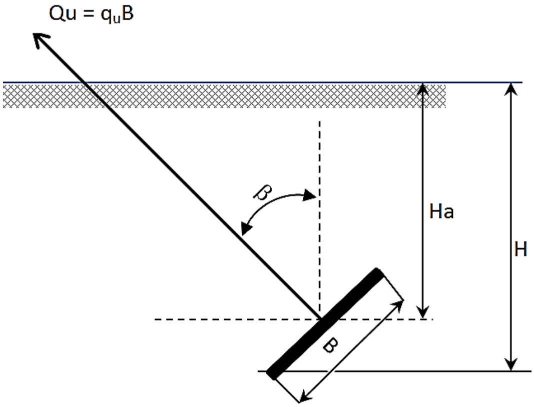


Figure 7.17 Geometric Parameters of Inclined Plate Anchors

Anchors located at $Ha/B < (Ha/B)_{\text{critical}}$ are called *shallow anchors,* and for this condition, at ultimate load, local shear failure in soil around the anchor takes place. As in the case of horizontal and vertical anchors, for a given anchor orientation (that is, i) there exists a critical average embedment ratio $Ha/B = (Ha/B)_{critical}$ beyond which the average breakout factor will remain practically constant, signifying *deep anchor behaviour.*

Estimates of the pull-out capacity of inclined anchors herein have been obtained by using the procedures developed by Lyamin and Sloan (2002), and Sloan and Kleeman (1995). For the case of a rough inclined anchor, the ultimate anchor pull-out capacity in cohesionless soil will be expressed as a function of the soil unit weight and embedment depth. For an embedment ratio of Ha/B, and $N\gamma$ is the break-out factor for a vertical anchor given by Fig. 7-18.

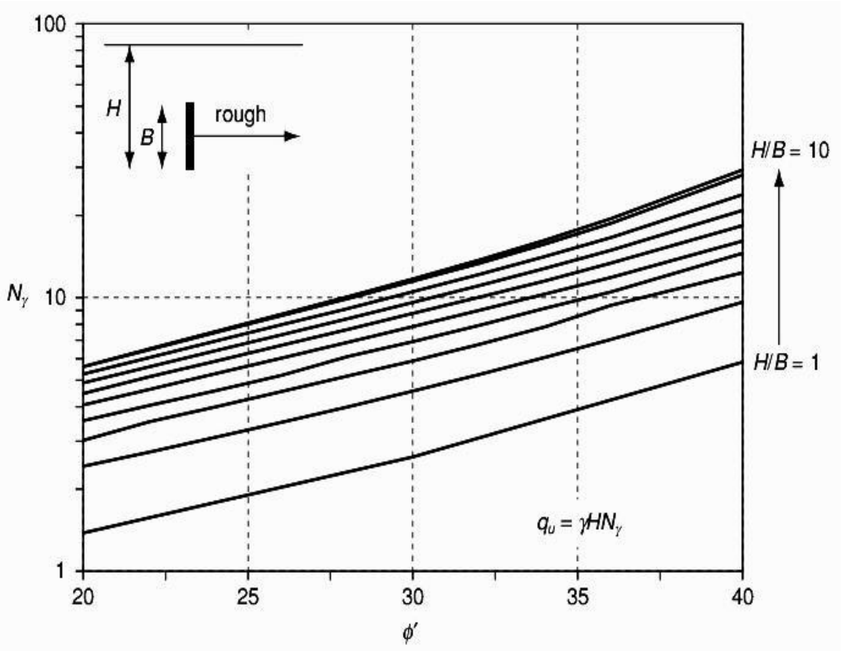


Figure 7.18 Break-out factors for rough vertical strip anchor in cohesionless soil

The ultimate capacity of a vertical anchor is given by equation,

$$q_u = \gamma.Ha.N\gamma \tag{7.5}$$

Where,

q_u = ultimate pull out capacity of plate anchor (per unit area)

$\gamma.Ha$ = effective overburden pressure on plate anchor

N_γ = break-out factor for vertical plate anchor (refer **Figure 7.18**)

The ultimate capacity of an inclined anchor the equ^n-7.5 is modified as

$$q_u = \gamma.Ha.N\gamma.\ i \tag{7.6}$$

Where, i is the inclination factor, shall be obtained from **Figure 7.19**.

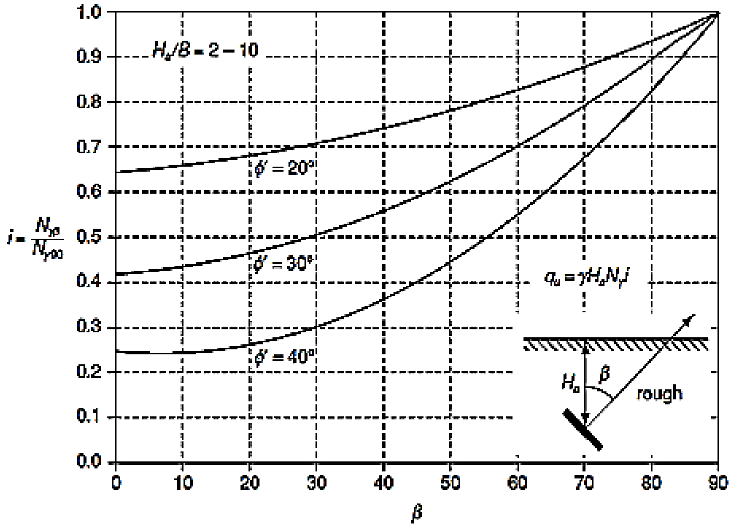


Figure 7.19 Inclination factors for inclined rough anchors in cohesionless soil: proposed design chart

Example 7.4

A plate anchor is installed to anchor a retaining wall. Check the pull out capacity of anchor with following design conditions.

i) ***Parameters given for design for anchor***

Total depth of anchor below G.L = 5.0 m

Length of anchor plate (L) = 2.0 m

Depth of anchor plate (B) = 2.0 m

ϕ of Backfill & Founding material = 32 deg.

Unit wt. of soil (γ') = 18.0 KN/m^3

a) Inclination of anchor from vertical plane β = 60 deg.

b) Inclination of anchor from vertical plane β = 90 deg.

Solution:

a) Pull-out capacity of horizontal anchor plate

H / B = 2.5

From figure 7-18 $N\gamma$ = 6.0

Horizontal pull-out capacity of anchor-plate

$Qu = \gamma . H . N\gamma$ = 540 KN/m^2

$Qu = qu \times L.B$ = 2160 KN

a) Pull-out capacity of inclined plate

Inclination of plate with vertical plane β = 60 deg.

Inclination factor *i* = 0.68

Pull-out capacity of inclined-plate

$Qu = \gamma . H . N\gamma . i$ = 367.2 KN/m^2

7.11 A-frame Anchor Pile

If any loose granular fill or soft cohesive soils strata exists up to a considerable depth behind the proposed retaining wall, the anchorage system should be supported by group of piles founded in firm strata. Lateral load applications in piles typically involve the application of significant lateral forces and/or moments on a deep foundation, resulting in significant internal bending moments and shear forces. **Fig (7-20 a & b)** shows an anchor block supported by a group of piles, these types of configuration, often referred to as “A-frame,” resist the lateral load through axial compression, tension, and shear through the piles.

Vertical piles are commonly used to resist vertical loads because they are easier and less costly to construct; however, batter piles typically provide the necessary resistance with less displacement and often with a smaller group of piles. The anchor pull force exerted by tie rod tends to induce compression in front pile and tension in rear pile. This type is employed where firm soil is available at large depth. These are used to resist lateral as well as vertical loads. Batter piles are installed at an angle (batter) that typically varies from 1H:12V to 1H:3V. It resists lateral loads primarily through the horizontal component of the axial load in the pile as determined by the angle of the batter, whereas vertical piles resist lateral loads primarily through structural shear and bending resistance.

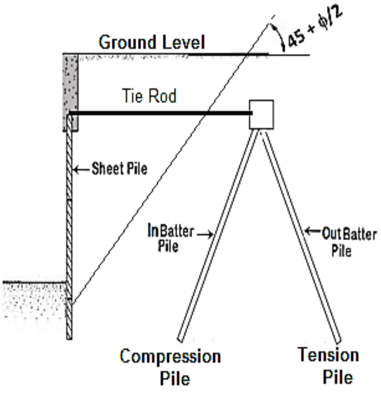


Figure 7.20 (a) : A-frame Anchor Pile with Batter Piles

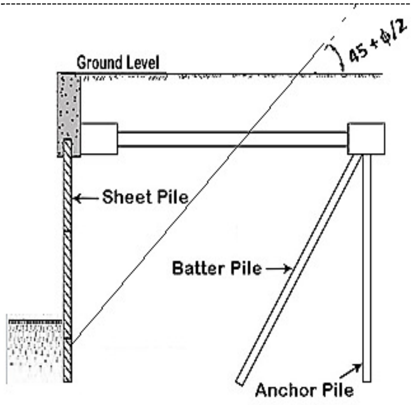


Figure 7.20 (b) : A-frame Anchor Pile in combination of Batter Pile & Vertical Pile

Bending of batter piles can also contribute to resistance of applied lateral loads, particularly for cases with small batter angle. The axial resistance in batter piles can be either in compression or tension.

7.11.1 Effect of Negative Skin Friction on Anchor Piles

Deep foundation elements installed through compressible soil strata can experience "down-drag" forces or negative skin friction along the shaft of pile which results from downward movement of adjacent soil relative to the pile. Negative skin friction results primarily from consolidation of a soft deposit caused the placement of fill. Negative skin friction is particularly severe on batter pile installations because the force of subsiding soil is large on the outer side of the batter pile and soil settles away from the inner side of the pile. This can result in bending of the pile. In such cases, batter piles with relieving platform provide optimal solution for anchored wall system. Relieving platform aids following advantages in piled supported platforms installed through compressible strata,

a) It eliminates the lateral earth pressure on wall due to live load surcharge;
b) Minimises or nullify the down-drag on batter piles;
c) It provides a safe and stable operating platform.

7.12 Anchorage of Wall Through group of Vertical Piles

Laterally loaded pile groups containing only vertical piles shall be designed based on lateral load analysis of individual vertical piles, assuming that the lateral load is evenly distributed among the piles. The distribution of vertical loads within both mixed and all vertical pile groups shall be determined based on elastic theory.

Driven piles in a group are considered to act individually;

- When the centre-to-centre spacing is greater than 3D in the direction perpendicular to the applied vertical loads, and;
- If spacing is greater than or equal to 8D parallel to the direction of horizontal loading.

For the case of closely-spaced piles in a group, the interaction behaviour is typically accounted for indirectly, using empirical procedures (e.g., Reese, et al, (1994) and Prakash and Sharma (1990)). One such procedure assumes a reduction in the coefficient of lateral subgrade reaction for a pile in a group from that of a single pile, using the ratios given in table 7.1 (after Prakash and Sharma, 1990].

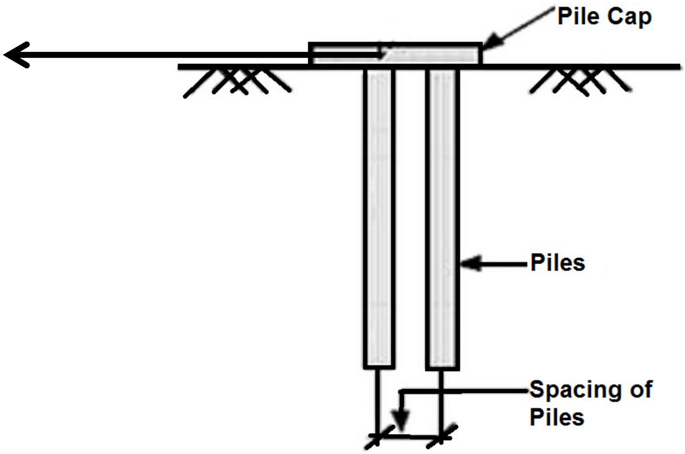


Figure 7.21 Anchorage through group of vertical piles

Table- 7.1: Ratio of Lateral Resistance of Pile in Group to Lateral Resistance of Single Pile (after Prakkash and Sharma, 1990)

Centre to Centre Pile Spacing (Parallel to the direction of applied load)	Ratio of Lateral Resistance of Pile in Group to Single Pile	
	Cohesive Soil	Cohesionless Soil
8D	1.00	1.00
6D	0.65	0.70
4D	0.50	0.60
3D	0.40	0.50

7.12.1 Design of Structural Anchor

Numerous methods are available to analyse the response of deep foundations subjected to lateral loads. One of the first analysis methods available was based on the concept of the subgrade modulus (Terzaghi 1955). In this method, which considers an elastic beam resting on an elastic foundation (Hetenyi 1946) the lateral reaction of soil against the pile is assumed to be dependent on the soil structure interaction.

Ground Anchor

7.13 Anchorage System with Tie Back or Ground Anchor

A ground anchor normally consists of a high tensile steel cable or bar, called the tendon, one end of which is held securely in the soil by a mass of cement grout or grouted soil: the other end of the tendon is anchored against a bearing plate on a retaining wall to be supported. The main application of ground anchors is in the construction of tie-backs for retaining walls. Ground anchors can be constructed in sands (including gravelly sands and silty sands) and stiff clays, and they can be used in situations where either temporary or permanent support is required. Ground anchors or tie-backs transmit tensile forces into the ground or rock. They are inserted into pre-drilled holes and bonded to the ground or rock by grout or other chemicals. Its load carrying capacity is generated as resisting reaction mobilized by stressing the ground along a specially formed anchorage zone.

7.13.1 Structural Details of Ground Anchor

Ground anchor is a structural element installed in soil that is used to transmit an applied tensile load into the ground. Grouted ground anchors are also referred to as “tiebacks”. The basic component of grouted ground anchor includes:

(1) anchorage; (2) free stressing (unbonded) length; and (3) bond length.

These and other components of a ground anchor are shown schematically in **Figure 7.22**.

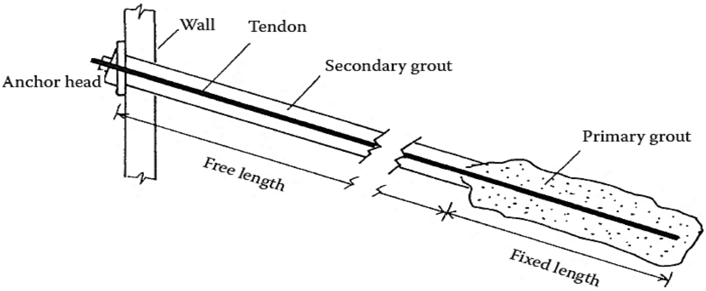


Figure 7.22 Basic components of Ground anchor

The anchorage is the combined system of anchor head, bearing plate, and trumpet that is capable of transmitting the pre-stressing force from the pre-stressing steel (bar or strand) to the ground surface or the supported structure. Anchorage components for a bar tendon and a strand tendon are shown in **Figure 7.23** and **Figure 7.24**, respectively.

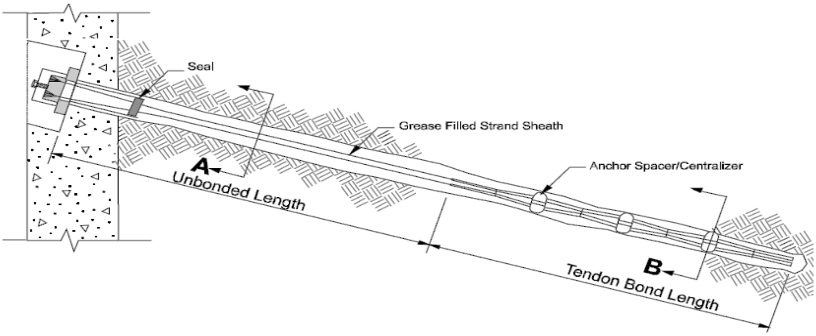

Seal
Grease Filled Strand Sheath
Anchor Spacer/Centralizer
A
Unbonded Length
B
Tendon Bond Length

Figure 7.23 Ground Anchor with Strand Tendon

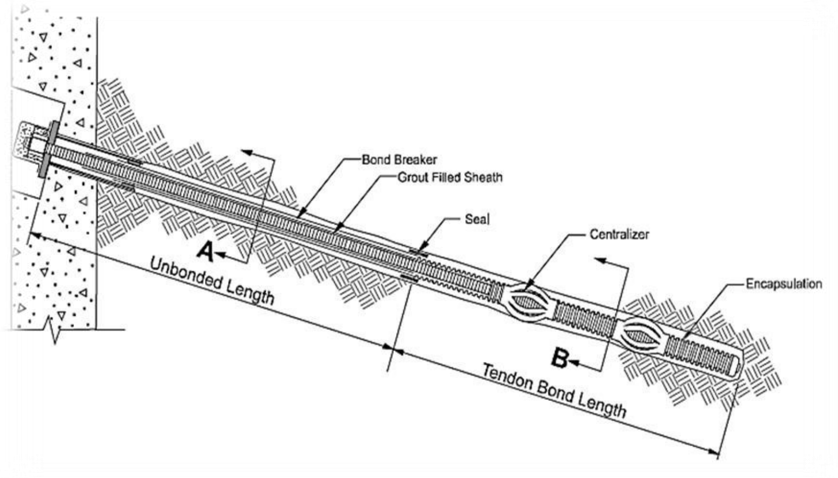

Bond Breaker
Grout Filled Sheath
Seal
Centralizer
Encapsulation
A
Unbonded Length
B
Tendon Bond Length

Figure 7.24 Ground Anchor with Bar Tendon

The **unbounded length** is the portion of the pre-stressing steel that is free to elongate elastically and transfer the resisting force from the bond length to the structure.

A **bond-breaker** is a smooth plastic sleeve that is placed over the tendon in the unbonded length to prevent the pre-stressing steel from bonding to the surrounding grout. It enables the pre-stressing steel in the unbonded length to elongate without obstruction during testing and stressing and leaves the pre-stressing steel unbonded after lock-off.

The **tendon bond length** is the length of the pre-stressing steel that is bonded to the grout and is capable of transmitting the applied tensile load into the ground. The anchor bond length should be located behind the critical failure surface.

A portion of the complete ground anchor assembly is referred to as the tendon. The tendon includes the pre-stressing steel element (strands or bars), corrosion protection, sheaths (also referred to as sheathings), centralizers, and spacers, but specifically excludes the grout. The definition of a tendon, as described in PTI (1996), also includes the anchorage; however, it is assumed herein that the tendon does not include the anchorage. The sheath is a smooth or corrugated pipe or tube that protects the pre-stressing steel in the unbonded length from corrosion. Centralizers position the tendon in the drilled hole such that the specified minimum grout cover is achieved around the tendon. For multiple element tendons, spacers are used to separate the strands or bars of the tendons so that each element is adequately bonded to the anchor grout. The grout is a Portland cement-based mixture that provides load transfer from the tendon to the ground and provides corrosion protection for the tendon.

Drilling methods for each of the three main soil and rock ground anchors include rotary, percussion, rotary/percussive, or auger drilling. The drilling method must not adversely affect the integrity of structures near the ground anchor locations or on the ground surface.

7.13.2 Soil and Rock Anchors

There are two types of ground anchors: (1) soil anchors, and (2) rock anchors. Ground anchors transfer load from tendon to grout then from grout to soil or rock. Each consists of a stressing tendon (rod or cable) connecting a fixed anchorage (within the soil or rock mass) to a surface anchor or head. Load transfer is by either friction along a straight shaft or by bearing against an under reamed bell or both. In cohesionless soils and rock, the fixed anchors are often formed by pressure grouting techniques, while in stiff cohesive soils, grout or concrete can also be placed using tremie methods, except where the inclination of the hole to the horizontal is not very large.

7.14 Selection of Soil Shear Strength Parameters for Design

7.14.1 General

As ground anchor can be constructed at any angle and in a wide variety of soils and rocks they are used for a multiplicity of purposes. One of their major uses is in the support of retaining walls; however, the basic concepts apply to all anchors. The simple view of a ground anchor is that it connects a man-made retaining structure to a natural and stable soil mass or rock strata. For this to be done successfully the nature and competence of the existing ground must be established. The data obtained from the ground investigations are used to estimate anchor capacity. The objective of this section is to provide guidance on selection of soil shear strength parameters for anchored system design.

7.14.2 Granular Soils

The drained shear strength of granular soil having sufficient permeability is conventionally represented by a drained effective stress friction angle. Because the undisturbed sampling of granular soil deposits is difficult, the representative friction angle for design may are generally estimated using the results of in situ tests such as SPT and CPT.

From laboratory test results, the particle size distribution has great significance in the design of anchors as the soil can be described according to the shape of the distribution curve, permeability and thus groutability can be assessed, and an indication of friction angle can be obtained. The effective particle size of a soil is governed by the size of the smallest 10% fraction of the soil. This size (D_{10}) is related to permeability and thus to permeability of the soil.

Soil permeability can be assessed from the particle size distribution curve. This is done by taking the square of the 10% fraction particle size (D_{10}) and assuming this to be the rate of flow.

For instance, for a D_{10} particle size of 0.2mm, permeability would be $(0.2)^2 = 0.04$ or $k=4 \times 10^{-2}$cm/sec.

To form a soil anchor grout bulb larger than the borehole diameter, it is necessary to force cement grout mass between the pores of soil grains. The limit for this is at an effective particle size of approximately 0.2 mm or fine sand. This represents a permeability of 10^{-1} to 10^{-2} cm/sec. Below this grain size the cement will not penetrate easily, but will densify the sand strata in the interfacing area of the bond zone to form a larger effective diameter than the borehole, but with a probable increase in the maximum ratio of 1.5R where R is borehole radius. Permeability of these soils would be $k=10^{-3}$ to 10^{-4} cm/sec. Soils with permeability less than $k=10^{-4}$ cm/sec and low cohesive strength indicate the presence of high proportions of silt. Such soils are not considered to be suitable materials in which to found ground anchors.

7.14.3 Shear Strength of Normally Consolidated Clay

The instability in soil under undrained conditions develops mainly due to contractive shear. The mechanism of soil deformation which attempts to mobilize frictional shearing resistance also causes the soil to want to contract under the prevailing confining stresses. This tendency to contract during shear is typical for normally to lightly over-consolidated soft to medium stiff clay. Since this tendency cannot be realized in fine grained soil due to its low permeability, positive pore water pressures are generated in the soil which reduce the effective stress and hence the mobilized frictional shearing resistance. In such cases the short-term undrained shearing resistance in the soil is less than drained condition, which would occur in long term. The short-term condition is critical for anchored walls constructed in normally to lightly over-consolidated clay.

In such cases, it is usual for soil samples (except sands and gravels) to be tested undrained in a tri-axial compression tester as this enables an examination of soil stability to be made without knowledge of pore water pressure being required. Also, as the effect, of wall and anchor construction and stressing on the soil structure is relatively quick, the "quick" undrained test most nearly represents actual site conditions. The minimum factor of safety thus occurs in the short-term undrained condition when strength is lowest. With time, excess pore water pressure induced by stress is dissipated and the soil structure stiffens and gains strength. Where soils of high plasticity or compressibility are encountered and are to be retained, their long-term consolidation characteristics should be determined. From this information, it can be established whether there will be a loss of anchor stress due to a dimensional change in the retained soil mass. Thus, the stress level below which the anchor will be stable can be decided, that is, where consolidation or creep will not occur. As failure at the ground-grout interface may be characterized by excessive deformations under sustained loading (i.e., creep). Soil deposits that are potentially susceptible to excessive creep deformations include:

i) organic soils;
ii) clay soils with an average liquidity index (LI) greater than 0.2;
iii) clay soils with an average liquid limit (LL) greater than 50;
iv) clay soils with an average plasticity index (PI) greater than 20.

Conservative anchor design loads and working bond stress values are recommended for design involving permanent anchor installations in such soils, unless based on results from a predesign or preproduction test program.

The liquid limit (LL), plastic limit (PL) and moisture content (w_n) and Liquidity Index (LI) of a clay soil are commonly measured clayey soil index properties. A low LI indicates that the moisture content is relatively close to the PL of the soil, indicating a potentially over-consolidated or stiff soil. A LI close to 1.0 indicates that the moisture content is relatively close to the LL for the soil, indicating a potentially normally consolidated or soft soil.

7.14.4 Other Considerations

- The soils should be further examined to establish whether a suitable bearing stratum exists that can be used for the construction of permanent anchors. Non-plastic soils and those of medium to low plasticity having high density together with granular materials such as silty sand or coarser can be considered suitable providing an adequate stratum is present.

- Anchors in plastic clays and silts, backfill materials or materials subject to normal consolidation only will probably not be successful anchorage subjects. Placing of the anchor bond zone in soils having a high organic content should be avoided. From this assessment, the utility of anchors can be decided.

7.14.5 Design Parameters for Over-consolidated Clay

In clay soils subjected to unloading conditions that result from excavation to form an anchored wall, the soil attempts to expand as it mobilizes frictional shearing resistance. This is resisted causing negative pore water pressure to be developed which increases the effective stress in the soil and hence increases the mobilized frictional shearing resistance. Thus, in over-consolidated clay subject to excavation, the short-term (undrained) strength and stability potentially exceeds that which would apply once drainage has occurred. Heavily over-consolidated clay soils are often fissured. Due to the fissured nature of over-consolidated clays, which can permit relatively rapid local drainage at the level of the discontinuities in the clay, it is generally difficult to define with any certainty the period of time during which the enhanced undrained shear strength of the clay may reliably be assumed to apply. Therefore, in over-consolidated clays, design analyses should be performed in terms of drained, effective stress parameters. Drained strength parameters for over-consolidated clays are discussed in chapter - 4.

7.15 General Design Concepts for Anchored Walls

The concept of an anchored wall system is to create an internally stable mass of soil that will resist external failure modes at an adequate level of serviceability. The design should limit the combined deformation in the wall and soil by providing a practical basis for construction. The design should consider the mobilization of resistance by both anchors and wall elements in response to loads applied to the wall system. Anchors should be capable to provide the required stabilizing forces, which can be transmitted back into the soil at a suitable distance behind the active soil zone, as illustrated in **Figure 7.25**. This requirement forms the basis for minimum distance behind the wall at which the anchor bond length is formed.

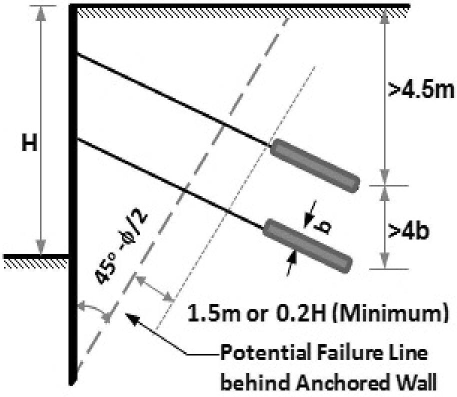


Figure 7.25 Potential failure conditions to be considered in design of anchored walls.

The anchor bond length must extend into the ground having stable soil or rock mass and does not envelope any potentially critical failure surfaces as illustrated in **Figure 7-26**. The installation depth of anchor and bond length shall be determined by the location of the deepest potential failure surfaces that have an insufficient factor of safety. For permanent works, the ground anchor should be treated as integral part of the system.

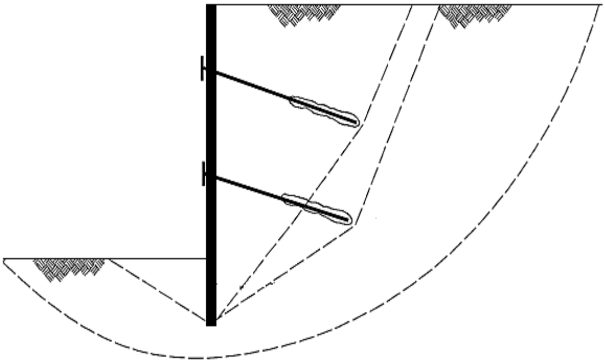


Figure 7.26 Potential failure planes behind retaining wall

7.15.1 Layout of Ground Anchor

Once the structural type of retaining wall is known, the number and levels of anchors can be considered. In marine bulkhead and water front structure, only one level of anchors is normally possible and they would be attached to a wale capping the retaining wall. For other walls in dry conditions, the number and tiers of anchors can be calculated by static analysis bearing in mind that soil anchor capacity is governed by the strata encountered and not by statics. As a general rule, the aim should be to construct anchors of maximum capacity within the limits imposed by soils and structural strengths as this is normally the most economical method. Other factors have to be considered such as the horizontal spacing of anchors which may be coinciding with sheet pile profiles. Spacing of anchor shall be made to coincide with lagging board width, H-Beam centres, and the king piles forming a contiguous wall, or the width of other type wall panels.

7.15.2 Ground Anchor Inclination

It is difficult to properly grout an anchor at an angle less than 15° although angles of 10 to 45 degrees are also in practice depending on capabilities of contractors. In addition, shallow anchor angles can lead to a lack of overburden depth which in turn limits the capacity of the anchor. Relative to this, it is desirable that a minimum of 4.5m of unsubmerged overburden be above the fixed anchor. Most soil anchors are installed at an angle of between 15° and 30°. However, when a suitable anchoring stratum lies at some depth, generally more than 9m, an angle of 45° may be chosen as a compromise between length of anchor and decrease in the resultant horizontal force for a given anchor capacity. It must be kept in mind, though, that by increasing the angle of inclination, the vertical component of the anchor load also increases, thus increasing the vertical load on the wall members and the underlying foundation material.

Regardless of the anchor inclination, the anchor bond zone must be developed behind potential slip surfaces and in soil or rock layers that can develop the necessary design load. Steep inclinations may be necessary to avoid underground utilities, adjacent foundations, right-of-way constraints, or weak soil or rock layers. Anchors should be installed as close to horizontal as possible to minimize vertical loads resulting from anchor lock-off loads, however grouting of anchors installed at angles less than 10 degrees is not common unless special grouting techniques are used. For an efficient anchor, a 15-degree inclination of the anchor should be assumed with a bond length of 12 m in soil or 8 m in rock.

7.16 Design for Internal Stability

7.16.1 Total Anchor Length

Ground anchors mobilize skin friction between the anchor bond zone and the ground. In general, this bond is dependent on the normal stress acting on the bond zone grout and the adhesion and friction mobilized between the ground and the grout. Anchors which are under-reamed may also develop the base resistance of the increased annular area.

In general, the ground-grout bond is mobilized progressively in uniform soil or rock as the stress is transferred along the bond length. Initially, as the anchor is stressed, the portion of the bond length nearest the load application elongates and transfers load to the ground. As the resistance in this portion of the bond length is mobilized, stress is transferred farther down. During this process, the anchor continues to elongate to mobilize deeper bond zones. Once the stress is transferred to the end of the bond zone and the ultimate ground-grout bond is exceeded, anchor failure by pull-out occurs.

Experience has shown that increasing the bond length for typical soil anchors beyond 9 to 12 m does not result in significant increases in resistance. A possible reason for this observation is that after the load has been transferred that distance down the bond zone, sufficient movement at the ground-grout interface has occurred in the upper bond length to decrease the upper ground-grout interface resistance to residual strength levels. A minimum unbonded length of 3 m for bar tendons and 4.5 m for strand tendons should be adopted. These minimum unbonded lengths are required to avoid unacceptable load reduction resulting from seating losses during load transfer and prestress losses due to creep in the prestressing steel or the soil. A minimum stressing length of 6m should be adopted to avoid unacceptable high prestress losses in anchors due to long-term relaxation, creep in steel and soil, and anchor seating losses. Wherever possible, free length should be *8-9m.* For rock anchors, typical bond lengths range from 3 to 10 m with a minimum of 3 m.

7.16.2 Stresses in Pre-stressing Steel Element

The design load shall not exceed 60 percent of the specified minimum tensile strength (SMTS) of the pre-stressing steel. The lock-off load shall not exceed 70 percent of the SMTS and the maximum test load shall not exceed 80 percent of the SMTS. For example, if the maximum test load is 133 percent of the design load, then the ground anchor should be selected based on a maximum allowable design load of (0.8/1.33) SMTS or 0.6 SMTS. If the maximum test load is 150 percent of the design load, then the maximum allowable design load is (0.8/1.5) SMTS or 0.53 SMTS.

7.16.3 Cement Grout Strength

Anchor grout for soil and rock anchors is typically a neat cement grout (i.e., grout containing no aggregate) although sand-cement grout may also be used for large diameter drilled holes. Pea gravel-sand-cement grout may be used for anchor grout outside the tendon encapsulation. A water/cement (w/c) ratio of 0.4 to 0.55 by weight and Type I cement will normally provide a minimum compressive strength of 30 MPa at the time of anchor stressing. For some projects, special additives may be required to improve the fluid flow characteristics of the grout. Admixtures are not typically required for most applications, but plasticizers may be beneficial for applications in high temperature and for long grout pumping distances.

7.17 Design of Bond Length

7.17.1 Estimated Capacity of Soil Anchors

Computation of the pull-out resistance, P_{ult} for tremie grouted anchors in cohesionless soils can be estimated from the following equation:

$$P_{ult} = \sigma'_z A_s L_s \alpha_g \qquad (7.7)$$

Where,

σ'_z = effective vertical stress at the midpoint of the load carrying length (Figure 7.25)

A_s = effective unit surface area of the anchor bond zone

L_s = effective length of the anchor bond zone (limited to about 8 m)

α_g = anchorage coefficient dependent on the soil type and conditions as given in Table 7.2

Table- 7.2: Anchorage Coefficient α_g

Soil Type	Relative Density		
	Loose	Compact	Dense
Silt	0.1	0.4	1.0
Fine Sand	0.2	0.6	1.5
Medium Sand	0.5	1.2	2.0
Coarse sand, Gravel	1.0	2.0	3.0

Computation of the pull-out resistance, for tremie-grouted anchors in stiff to very hard clay soils can be estimated from the following equation:

$$P_{ult} = \alpha_c A_s L_s S_u \tag{7.8}$$

Where,

α_c = reduction factor related to the undrained shear strength (Figure 7.28)

A_s = effective unit surface area of the anchor bond zones

L_s = effective length of the anchor bond zone (limited to about 8 m)

S_u= average undrained shear strength of the clay over the anchor length,

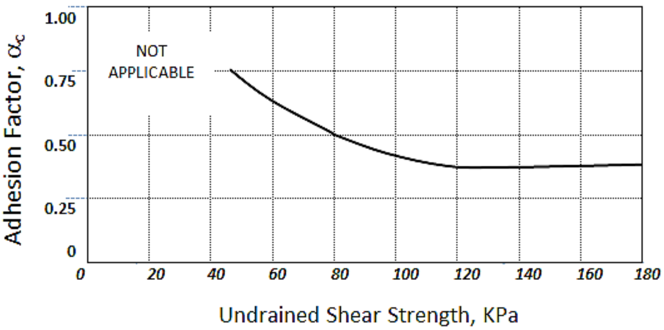


Figure 7.28 Adhesion Factor for Anchors Installed in Cohesive Soils

Normally, anchors should not be formed in soft or firm clays, sensitive clays, or in uncontrolled fill materials because of the large deformations that can occur, both at and subsequent to loading. Small diameter, pressure grouted anchors can develop higher capacities than those that could be calculated using the methods above for an equivalent diameter tremie-grouted anchor.

Preliminary capacities of pressure grouted anchors may be calculated according to the values provided in Table 7.3 provided that:

- the nominal diameter of the anchor is between 150 mm and 200 mm;
- grout is injected using a positive pressure of about 1 MPa, and
- The centre to centre spacing of the anchors in the bond zone is more than 4 times the anchor diameter or 20 % of the bond zone length (see also Figure 7.27).

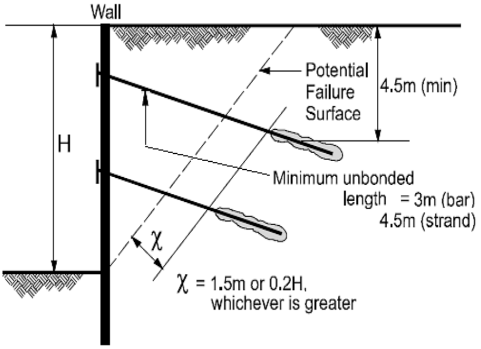

Wall
H
Potential
Failure
Surface
4.5m (min)
Minimum unbonded length = 3m (bar)
4.5m (strand)
χ
χ = 1.5m or 0.2H,
whichever is greater

Figure 7.27 (a)

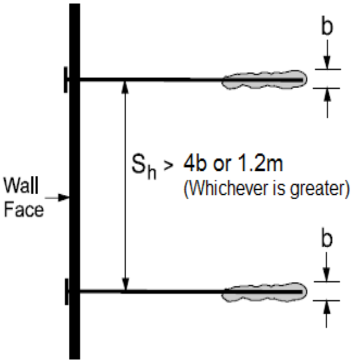

b
Wall
Face
S_h > 4b or 1.2m
(Whichever is greater)
b

Figure 7.27 (b)

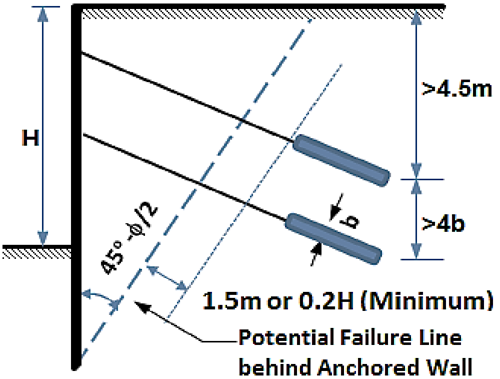


Figure 7.27 (c)

Figure 7.27 Minimum spacing and depth for soil anchors

The ultimate load transfer values provided in Table 7.3 are generally suitable for anchors constructed using a single stage of grouting. Some anchors, however, are designed and constructed such that they can be post grouted in one or more stages subsequent to initial grouting. During secondary and later stages of grouting, the initial grout around the anchor zone is fractured and the secondary ground forces the fractured grout against the soil, and the secondary grout can also further permeate or compact the surrounding ground.

Table 7.3 Estimation of Capacity for Pressure Grouted Anchors (After FHWA 1984)

Ground type	**Relative Density/ consistency (SPT "N" Range)**	**Estimated Ultimate Load Transfer (KN/m)**
Coarse Sand and Gravel	*Loose (4 - 10)*	*145*
	Medium Compact (10 - 30)	*220*
	Compact (30 - 50)	*290*

Medium Sand	*Loose (4 - 10)*	*100*
	Medium Compact (10 - 30)	*145*
	Compact (30 - 50)	*190*
Sand and Silt	*Loose (4 - 10)*	*75*
	Medium Compact (10 - 30)	*100*
	Compact (30 - 50)	*130*
Low-Plasticity Silt & clay	*Stiff (10 - 20)*	*30*
	Hard (20 - 40)	*60*

Multiple stages of grouting can increase the load transfer values above those provided in Table 7.3, however, the use of higher values for such anchors should be based on detailed experience with local ground conditions and construction practices. When considering pressure grouted anchors, consideration should be given to the actual grouting pressures, pressure dissipation around the anchor, and the potential for heaving or fracturing the ground.

Example 7.5

A ground anchor is required to be installed through clayey strata at an inclination of 20 deg to transfer 70 KN/m horizontal force from the head of a wall. The cohesion value of soil, Su is 100 Kpa. Calculate the fixed length of anchor required to counter the pull-out force, if horizontal spacing of anchor is 1.80m.

Solution

Horizontal force on wall F = 70.0 KN/m

Cohesion value of soil Su = 100 Kpa

Type of soil = Stiff Clay

Effective dia. of anchor d = 0.3 m

Angle of inclination of anchors α = 20 deg

Factor of safety for anchor (F.O.S) = 2.5

Horizontal spacing of anchors = 2.0 m

Horizontal force on each anchor P = 140.0 KN

(T = F x 2.0)

Working force along the anchor

(T / cos 20) = 149.0 KN

Ultimate capacity of anchors

(T / cos α) x F.O.S = 372.46 KN

In clayey strata ; $T_{ult} = \alpha_c . A_s . L_s . S_u$

For, 100 Kpa Stiff Clay α_c = 0.4 (from Fig 7.28)

(As = $\pi \, . \, d$) As = 0.94 m

Fixed length of anchor required

(Ls = T_{ult} /α_c . As . Su) = 9.9 meter

Provide, fixed length of anchor as = 10.0 meter

Example 7.6

A ground anchor is installed through dense granular strata at an inclination of 30 deg from the horizontal to transfer 380 KN/m horizontal force from the head of a wall. The φ- value of soil is 36 deg. Calculate the fixed length of anchor required to counter the pull-out force, if horizontal spacing of anchor is 2.0m.

Solution:

Horizontal force on wall F = 250.0 KN/m

ϕ-value of soil = 36 deg

Average bulk density of soil = 17.6 KN/m^3

Type of soil = Coarse sand

Effective dia. of anchor d = 0.3 m

Angle of inclination of anchors α = 30 deg

Depth of C.G of anchor = 9 m

Effective overburden pressure σ_z = 88.4 KN/m^2

(2.0 x 17.6 + 7 x 7.6)

Factor of safety for anchor (F.O.S) = 2.5

Horizontal spacing of anchors = 2.0 m

Horizontal force on each anchor P = 500.0 KN

(T = F x 2.0)

Working force along the anchor

(T / cos 30) = 577.4 KN

Ultimate capacity of anchors -T_{ult}

(T / cos α) x F.O.S = 1443.4 KN

In granular strata; $T_{ult} = \sigma'_z . A_s . L_s . \alpha_g$

For, ϕ = 36deg (Coarse sand) α_g = 2.0 (from Table-7.2)

(As = $\pi \, . \, d$) As = 0.94 m

Fixed length of anchor required

(Ls = T_{ult} /σ'$_z$. α_g. As) = 8.7 meter

Provide, fixed length of anchor as = 9.0 meter

Example 7.7

A ground anchor is installed through dense granular strata at an inclination of 30 deg from the horizontal to transfer 380 KN/m horizontal forces from the head of a wall. The φ- value of soil is 36 deg. Calculate the fixed length of anchor required to counter the pull-out force, if horizontal spacing of anchor is 2.0m.

Solution:

Horizontal force on wall F = 250.0 KN/m

ϕ-value of soil = 36 deg

Average bulk density of soil = 17.6 KN/m^3

Type of soil = Coarse sand

Effective dia. of anchor d = 0.3 m

Angle of inclination of anchors α = 30 deg

Depth of C.G of anchor = 9 m

Effective overburden pressure σ_z = 88.4 KN/m^2

(2.0 x 17.6 + 7 x 7.6)

Factor of safety for anchor (F.O.S) = 2.5

Horizontal spacing of anchors = 2.0 m

Horizontal force on each anchor P = 500.0 KN

(T = F x 2.0)

Working force along the anchor

(T / cos 30) = 577.4 KN

Ultimate capacity of anchors -T_{ult}

(T / cos α) x F.O.S = 1443.4 KN

In granular strata; $T_{ult} = \sigma'_z . A_S . L_S . \alpha_g$

For, ϕ = 36deg (Coarse sand) α_g = 2.0 (from Table-7.2)

(As = $\pi \, . \, d$) As = 0.94 m

Fixed length of anchor required

(Ls = T_{ult} /σ'$_z$. α_g . As) = 8.7 meter

Provide, fixed length of anchor as = 9.0 meter

7.17.2 Estimated Capacity of Sand Anchors

In considering the loads obtainable from sand strata, it is necessary to obtain accurate soil data, including sieve analysis grading curves, angles of internal friction and strata thickness. Where this description indicates that the soil permeability would be k=10^{-1} to 10^{-2} cm/sec, the fixed anchor formed where the grout pressure is a nominal 10 to 40 psi [0.00 69N/ mm^2] (i.e. . the hydrostatic head dependent upon a drilled depth) would not consist of a smooth grout cylinder since the sand would permit some permeation by a very fluid cement grout. Field trials have enabled an empirical rule to be established which is as follows:

$T_{ult} = L\,N'\tan\phi$ (7.9)

Where,

T_{ult} = Ultimate load capacity of the anchor- (KN)

L = Fixed length of anchor (meter)

N' = 420 to 600 KN per- meter

ϕ = Angle of internal friction

However, the use of pressure grouting techniques means that increased anchor loadings can be expected by virtue of the fact that greater penetration of the cement grout into the surrounding soil is achieved, consolidation or densification of the soil takes place and there is a residual "locked-in" grout pressure remaining after completion of pressure grouting in the fixed anchor length.

A further empirical formula has been derived from field trials to express this, using the factors above:

Where, $'T_{'ult} = p'\,\pi \,.\, d \,.\, L \,.\, \tan\phi$ (7.10)

T_{ult} = Ultimate load capacity of the anchor

P' = Grout pressure at 2 psi ($0.014 N/mm^2$) per foot of over-bur-den above the top of fixed anchor taken as an average over the fixed anchor length.

d = Effective diameter of fixed anchor

L = Fixed anchor length

The effective diameter of the grout bulb, d, is estimated from the soil permeability with rates of 10^{-1} to 10^{-2} cm/sec indicating soil infiltration by the grout and 10^{-3} to 10^{-4} cm/sec for soil densification, local to the borehole. Grout takes determined during construction can be a good guide to grout bulb size and eventual anchor performance.

The capacity of anchors estimated using the methods above presumes a relatively linear increase of capacity with a corresponding increase in bond zone length. However, anchor capacities generally do not increase once the length of the bond zone increases beyond about 8 m.

The presumptive ultimate values of load transfer for preliminary design of small diameter straight shaft gravity-grouted ground anchors in soil are given in Table 7.3, which are average ultimate bond stress for ground/grout interface along anchor bond zone (after PTI, 1996). Anchor bond lengths for gravity-grouted, pressure-grouted, and post-grouted soil anchors are typically 4.5 to 12 m since significant increases in capacity for bond lengths greater than approximately 12 m cannot be achieved unless specialized methods are used to transfer load from the top of the anchor bond zone towards the end of the anchor.

The performance of soil and rock anchors depends not only on minor variations in soil and groundwater conditions, but also on construction techniques and details. Anchor capacities calculated using the procedures outlined above represents a rational basis for design, but it must be verified by field test or proof loading during construction.

Example: 7.8

A ground anchor (tie-back) is installed at an inclination of 30 deg. from the horizontal to transfer 150 KN/m horizontal forces from the head of a retaining structure. The bond length of anchor is installed in medium dense silty sand having φ- value as 32 deg. Calculate the fixed length of anchor to be provided to counter the pull-out force, if horizontal spacing of anchor is 1.80m.

Solution:

Horizontal force on wall F = 150.0 KN/m

ϕ- value of soil = 32 deg

Type of soil (Med. fine silty sand)

Angle of inclination of anchors α = 30 deg

Factor of safety for anchor (F.O.S) = 2.5

Horizontal spacing of anchors = 1.8 m

Horizontal force on each anchor P = 270.0 KN

(T = F x 1.8)

Working force along the anchor

(T / cos 30) = 311.8 KN

Ultimate capacity of anchors

(T / cos α) x F.O.S = 779.42 KN

In sandy strata; **T_{ult} = L N' tan ϕ**

For, Med. fine silty sand N` = 145.0 KN/m
tan ϕ = 0.62
Fixed length of anchor required
(L = T_{ult} / N' tan ϕ) = 8.6 meter
Provide, fixed length of anchor as = 9.0 meter

Alternatively, the value of N' tanϕ, (ultimate load transfer capacity of anchor) can be replaced by a direct value from Table -7.3.

For, Medium Compact fine sand
τ_{ult} for SPT 'N' = (10 - 30) = 100 KN/m
L = T_{ult} / τ_{ult} = 7.8 meter
Provide, fixed length of anchor as = 8.0 meter

Example: 7.9

A ground anchor is installed at an inclination of 30 deg from the horizontal to transfer 380 KN/m horizontal forces from the head of a wall. The anchor is installed through medium dense coarse sand having φ- value as 34 deg. Calculate the fixed length of anchor required to counter the pull-out force, if horizontal spacing of anchor is 1.80m.

Solution:

Horizontal force on wall F = 380.0 KN/m
ϕ- value of soil = 34 deg
Type of soil = Coarse sand
Angle of inclination of anchors α = 30 deg
Factor of safety for anchor (F.O.S) = 2.5
Horizontal spacing of anchors = 1.8 m
Horizontal force on each anchor (T = F x 1.8) = 684.0 KN

Working force along the anchor

(T / cos 30) = 789.8 KN

Ultimate capacity of anchors

(T / cos α) x F.O.S = 789.8 x 2.5 = 1974.5 KN

In sandy strata, $T_{ult} = L\,N'\tan\phi$

For, Coarse sand N\` = 400.0 KN/m

$\tan\phi$ = 0.67

Fixed length of anchor required

[$L = T_{ult} / N'\tan\phi = 1974.5 / (400 \times 0.67)$] = 7.3 meter

Provide, fixed length of anchor as = 8.0 meter

Alternatively, the value of N' tanϕ, (ultimate load transfer capacity of anchor) can be replaced by a direct value from Table -7.3.

For, Medium Compact coarse sand

For SPT 'N' = (10 - 30) τ_{ult} = 220 KN/m

$L = T_{ult} / \tau_{ult}$ = 9.0 meter

Provide, fixed length of anchor as = 9.0 meter

7.18 Rock Anchors

Anchorage design in rock is based on an allowable grout to rock bond stress acting over the fixed anchorage length. The allowable bond stress should be smaller than *1/30* times the unconfined compressive strength of the grout. It should not exceed 1300 KPa. Whenever possible, the capacity of an anchor in rock should be established by means of a pull-out test. Table 7.5 provides estimates of ultimate load transfer values for a variety of rock types.

TABLE 7.4 *Estimation a/Capacity/or Pressure Grouted Anchors (after FHWA 1984)*

Rock Type	**Estimated Ultimate Load Transfer (KN/m)**
Granite or Basalt	730

Dolomitic Limestone	580
Soft Limestone	430
Sandstone	430
Slates and Hard shales	360
Soft Shales	145

The allowable anchor load is determined by dividing the ultimate capacity of the anchor by a factor of safety. Where no pull-out tests are carried out, the allowable anchor load is commonly obtained by dividing the computed capacity of the anchor by a factor of safety of 3 or more.

Example: 7.10

A rock anchor is installed through basalt rock strata at an inclination of 40 deg from the horizontal to transfer 245 KN/m horizontal force from the head of a wall. The φ- value of soil is 36 deg. Calculate the fixed length of anchor required to counter the pull-out force, if horizontal spacing of anchor is 2.0m.

Solution:

Design of Rock Anchor

Horizontal force on wall F = 245.0 KN/m

Type of Rock = Basalt

UCS of rock = 300 Mpa

UCS of Grout material = 30 Mpa

Angle of inclination of anchors α = 40 deg

Dia. of drilled shaft in rock (fixed length) d = 100 mm

Factor of safety for anchor (F.O.S) = 3.0

Horizontal spacing of anchors = 2.5 m

Horizontal force on each anchor T = 612.5 KN

(T = F x 2.5)

Working force along the anchor

(T / cos 40) = 799.6 KN

Ultimate capacity of anchors T_{ult} =

T_{ult} = (T / cos 40) x FOS = 2398.7 KN

In rock strata; $\pi \,.\, d.\, L \,.\, \tau$ T_{ult} =

For, Basalt ultimate bond strength

$\tau = 0.2 \text{ x } (UCS)^{0.5}$ = 3.46 Mpa

Or,

1 / 30 of grout strength = 1.00 Mpa

Design bond strength at rock-grout interface

Min (3.46, 1.0) = 1.00 Mpa

Fixed length of anchor required (L)

($L = T_{ult} / \pi \,.\, d.\, \tau$) = 7.6 meter

Provide, fixed length of anchor; L = 8.0 meter

7.18.1 Estimated Capacity Rock Anchors

For rock anchors, typical bond lengths range from 3 to 10 m with a minimum of 3 m. The maximum allowable anchor design load in competent rock may be determined by multiplying the bond length by the ultimate transfer load and dividing by a factor of safety of 3.0.

In the softer rocks, anchors would be formed in the same configuration as that shown in **Figure 7.29**. Tomlinson (1987) proposed that the calculation of the volume of rock ***V,*** in a single cone with a half-angle of 30 at various angles of inclination **0** to a horizontal rock surface. The effect of overlapping cones of rock in groups of vertical or raking anchorages can be calculated. Where overburden overlies the rock, the 'cones' should not be continued through the overburden. The contribution to uplift restraint provided by the overburden can be considered as consisting of a cylindrical body normal to the ground surface and coincident with the limits of the cone at rock-head. The rock depth h, may be taken to the top or mid-point of the fixed anchor where the load is transferred by bond. Alternatively, where load is transferred by end plate at the distal end of the fixed anchor, the full anchorage depth within the rock may be acceptable. For inclined anchorages, h, is measured along the axis of the anchorage.

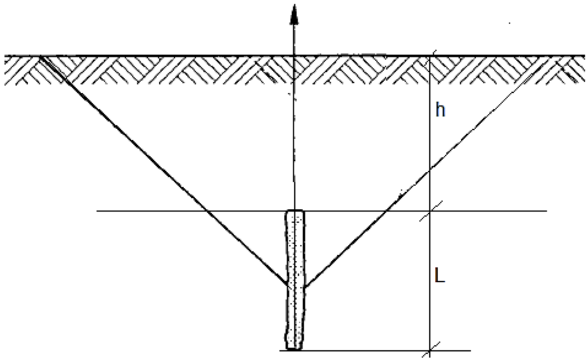


Figure 7.29 Interaction of Cone and grouted hole for overall stability analysis of rock anchor

However, ultimate capacity is not governed by cohesive failure or plastic flow as in clay soils, but by crushing strengths of rock and grout, the grout/rock interface bond strength and the bond between the tendon steel and grout. In massive type rock with few bedding planes, it may

7.19 Safety Factors for Anchors

7.19.1 FHWA

Once the theoretical anchor load is determined in the above manner, it is then necessary to apply, the proper factors of safety to the various components of the retaining wall. For temporary, anchors in coarse grained soils, a minimum safety factor of 2.0 should be used. In the case of permanent soil anchors or the structure is an important one where, serious economic loss or loss of life is likely as a result of a failure, and/or where corrective measures would be extremely expensive or impossible, adoption of a factor of safety of 2.5 is advisable.

7.19.2 Canadian Geotechnical Design Manual

The capacity of an anchor in soil can also be established by a pull-out test. According to Canadian Geotechnical Design Manual, where no pull-out tests are carried out, the allowable anchor load is commonly obtained by dividing the computed capacity of the anchor by a factor of safety of 3 or more.

7.19.3 BS 8081

Table- 7.6 : Minimum safety factors for design of Individual Anchorages

Anchorage Category	**Minimum Safety Factors**			
	Tendon	**Ground/ Grout Interface**	**Grout/ Tendon interface**	**Proof Load**
Temporary anchorages where a service life is less than six months, and failure of anchor would not cause serious consequences.	1.4	2.0	2.0	1.10
Temporary anchorages with service life less than two years, and there is no danger to public safety without adequate warning	1.6	2.5	2.5	1.25
Permanent anchorages where risk of corrosion is high and consequences of failure is serious.	2.0	3.0	3.0	1.50

7.20 Design Tests

Design tests are carried out before the installation of working (production) anchors. They provide criteria to substantiate the design parameters used to define acceptable performance of service anchors during proof testing. In general, a cyclic loading pattern for design tests should be adopted as follows:

7.20.1 Stressing and Proof Loading of Anchors

The method of tensioning, the sequence of stressing and the procedures to be adopted for each stressing operation should be specified at the planning stage of a project. For grouts based on Portland cement, stressing should not commence until the primary grout forming the fixed anchor has attained uniaxial compression strength of at least 30 MPa, as verified from tests on 100 mm or 150 mm cubes. No tendon that is intended to form part of any temporary or permanent work should be stressed at any time beyond 80 % of the guaranteed ultimate strength.

7.20.2 Proof Tests

Proof tests, carried out on all working (servicing) anchors, can employ procedures similar to design tests, but are normally abbreviated and simpler. Proof tests are not intended to be destructive. A proof test should be taken to the maximum test load of 1.33 times the working (service) load.

Upon reaching and having maintained the maximum load, the tension is released to the specified "lock-off Load" and the tension load is transferred from the jack to the nut/anchor head, whereupon the jack and equipment are removed. If an anchorage fails to meet the specified acceptance criteria, further stages of testing may be required.

7.20.3 Failure of Grout-Tendon Bond

The bond between the grout and steel tendon must not be exceeded if the full strength of the supporting ground is to be mobilized. The bond resistance between grout and steel is a function of compressive strength of grout, which increases with increase in compressive strength of the grout.

7.20.4 External Stability of Ground Anchor

Conventional limit equilibrium methods for slope stability can be used to evaluate the external stability of an anchored wall system. An anchored wall system is externally stable if potential slip surfaces passing behind or through the anchors have a factor of safety that exceeds the target factor of safety.

For temporary anchored systems constructed in soft to medium stiff clay soils, external stability should be evaluated using short-term (i.e., undrained) strength parameters and temporary loading conditions. For permanent anchored systems constructed in soils, external stability for both short term and long-term conditions should be checked. For systems constructed in stiff clays, external stability for short-term conditions may not be critical, but long-term conditions, using drained shear strength parameters, may be critical.

A minimum acceptable factor of safety for external stability is 1.3 for temporary applications. For permanent applications that are critical, a higher factor of safety (e.g., 1.5) may be used.

7.21 Installation of Anchors

The advancement of the hole for a soil or rock anchor must be carried out in a manner that precludes the possibility of loss of ground or flow of wet soil into the hole. Where water bearing

zones or wet soil are encountered, holes must be temporarily cased, or supported by other means. In some rock formations and in soil, small diameter anchors are often installed using flush joint steel casing with an internal drill bit and a cutting shoe at the advancing end of the casing. Anchors in soil are sometimes installed by drilling a hole using a hollow stem auger to the full anchor depth. Where the size of hole is of 200 mm diameter or less, grout is injected through the casing or hollow auger stem (often at a pressure greater than 10 MPa) to achieve a grouted anchor length.

Care must be taken to ensure that high grout pressure will not cause damage to adjacent structures or services.

Where the hole is up to 300 mm diameter, concrete rather than grout is often pumped through the casing or hollow stem auger as it is withdrawn from the hole. Normally, if a hole is of this larger diameter, a lesser pressure to inject the concrete is used. Depending on the nature of the ground and the inclination of the hole, the use of hollow stem augers for anchor construction may result in significant disturbance to the ground surrounding the hole. If auger rotation, penetration, and extraction rates are not well controlled, excess ground may be withdrawn around the auger stem and wet, loose, or soft soils may cave toward the auger and be removed during anchor drilling. Such disturbance can result in low capacities, excessive deformation, or anchor failure and installation methods must be chosen appropriate to the ground conditions.

7.22 Soil Anchor Capacity Established From Pull-Out Tests

Where the capacity of anchors is to be determined by pull-out tests, at least one anchor in 100 of those actually used in the project should be tested - with a minimum of one in each soil or rock type. If the anchor movement is appreciable, the capacity of the anchor is defined as that load at which the anchor begins to pull out of the ground (plastic displacement). If the load is not clearly apparent from the test data, the capacity is taken as the maximum load at which the observed movement is still tolerable for the structure. If the ultimate resistance is not reached, or no plastic displacement is observed in the test loading, the greatest applied test load should be assumed as the capacity for calculation of the allowable anchor.

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Chapter 8

Durability of Steel Piles

Steel doesn't know how old it is. Their condition of exposure states, how long will they last.

- *Anonymous*

8.1 Introduction

Corrosion is a natural decaying process based on electro-chemical reaction that affects all metals to some degree as shown in **Figure-8-1**. It is the gradual destruction of materials (usually a metal) by chemical and/or electrochemical reaction with their environment. As the steel corrodes, it loses thickness and degrades the useful sectional properties of structures, its strength, appearance and permeability to the liquids. Steel corrosion becomes a design concern when the projected loss of steel thickness over the design life impacts the structural capacity of the piling.

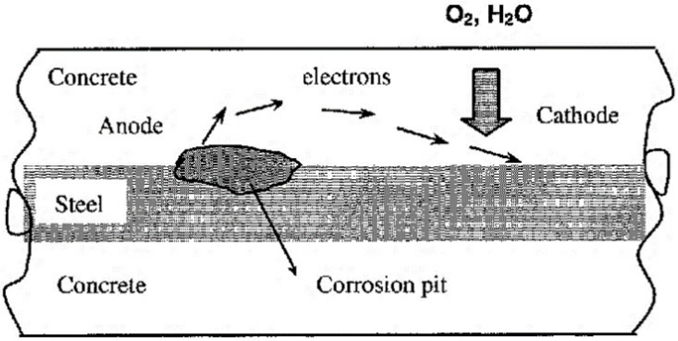


Figure 8.1 An Indicative Process of Corrosion Process in Steel

8.2 Corrosion Environments

Although many factors can contribute to the corrosion process, the most important are moisture, oxygen and chlorides (principally salt from seawater).

Moisture is perhaps the most important element necessary for corrosion. It acts as an electrolyte, allowing electrons to move between the anode and cathode. For this reason, pilings subject to atmospheric exposures tend to have much lower corrosion rates than those in wet environments.

The second major factor is **oxygen**, which stimulates the cathodic reaction in the presence of moisture.

Chlorides, primarily found in marine environments, are the third major factor affecting steel sheet piling corrosion. Chlorides increase the electrical conductivity of the water, allowing corrosion to proceed at a higher rate. Theoretically, factors such as the electrical resistivity of the soil or water, and the soil/water pH can impact corrosion.

In most cases, the rate of steel sheet piling corrosion tends to decrease over time, due to the formation of rust and fouling in marine environments, both of which have protective properties. In exposures where the protective layer of rust is removed, such as by wave action in the splash zone of marine environments, corrosion rates do not decrease over time. Hence, these areas tend to have the highest overall corrosion rates. Other conditions that remove the protective layer, such as abrasion due to propeller wash, or that may change the exposure conditions, such as erosion, also need to be considered in the overall corrosion evaluation.

Steel sheet piles used in close proximity of marine environment or near saltwater are also susceptible to corrosion due to higher concentrations of salt in the air. Water with a pH of 7.2 to 8.2 is more likely to expedite corrosion.

For unprotected steel piles, allowances for corrosion should be applied to each face of the pile exposed to the specific environment. When considering the loss of section for a sheet pile, the nature of the structure is such that different corrosion conditions can apply to the front and rear faces of the pile. This is best demonstrated by considering a marine wall where the front face of the wall is exposed to seawater and tidal action but the back face may be in contact with soil as shown in **Figure-8-2**

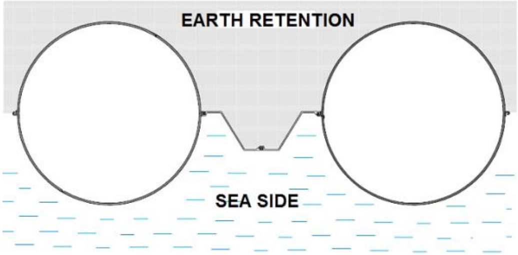


Figure 8.2 Different Corrosion Environments in Retaining Structure

In such situations, assessment of corrosion loss is made by summing the corrosion effects for each face of the pile at any given elevation.

Free-standing isolated piles, such as H or tubular load bearing piles will normally be exposed to the same environment around their entire perimeter at any given elevation. In case of unfilled open tubular piles, the potential for corrosion on inner face of the tube should also be accounted.

8.3 Corrosion in Naturally Occurring Soils

The corrosion of steel piles driven into undisturbed soils is negligible, irrespective of the soil characteristics and compositions. The minimal corrosion attack is due to very low dissolved oxygen concentration present in undisturbed soils and the very slow transport (by diffusion) of that dissolved oxygen through the pore structure of the soil to the steel surface even in the presence of ground water.

Most corrosion takes place above the water table. At the water table, some corrosion may sometimes occur if the water is mobile and replenished with oxygenated waters. Even in these circumstances, the water content of compacted natural soils is very low and corrosion is most likely in the form of isolated pitting in the water table zone.

Soil with a pH of 7 or greater is more acidic and will therefore be more corrosive to steel.

EN 1993-5 recommends a corrosion rate of 0.012 mm/year/side for piles driven into undisturbed soils.

8.4 Corrosion in infill and disturbed soils

The area, where excavation and backfilling process disturbs the soil, the situation can be different. In such cases filled up soil can get aerated and may include contaminants. However, once the filled up soil get compacted, oxygen replenishment will not be possible, and the corrosion process will slow down naturally.

According to EN 1993-5, long-term corrosion rates of steel in filled-up soils placed in non-aggressive and compacted state is approximately 0.02 mm/year/side. For aggressive and non-compacted fills, long-term corrosion rate is approximately 0.06 mm/year/side.

8.5 Corrosion in water

8.5.1 Fresh water environment

The term ‘fresh water’ distinguishes these from sea or estuarine brackish water with high salt concentration. The corrosion of steel in fresh water depends upon the type of water, although pH has little effect over the range pH 4 to pH 9, which covers the majority of natural waters. A corrosion loss of submerged steel in fresh water is generally lower than for seawater.

EN 1993-5 gives the worst-case corrosion rate for fresh waters of 0.014 mm/year/side for temperate climates.

8.5.2 Marine environments

Marine environments normally encompass several different exposure zones and each one requires individual consideration, mostly due to the variation of water level and/or dissolved oxygen concentration. The environments where port and harbour steel structures are applied are roughly classified into five zones, namely, atmospheric zone, splash zone, tidal zone, submerged zone, and mud zone. In cases where long steel products such as steel sheet pile and steel pipe pile extend through multiple environments (tidal zone, submerged zone, and mud zone), rate of corrosion are different according to the prevailing environment. The area where serious problem arises regarding the corrosion of steel is the splash zone and just above the mean high water level (M.H.W.L.). Corrosion tendencies by corrosive environment are indicated in **Figure 8.3**.

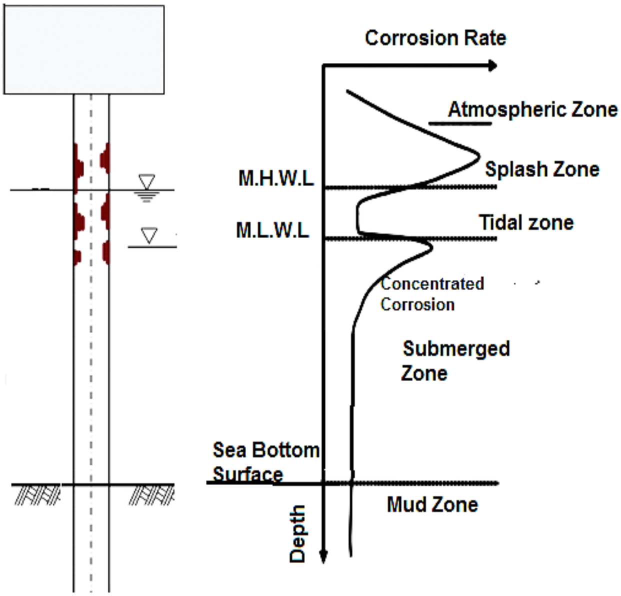

Corrosion Rate
Atmospheric Zone
Splash Zone
M.H.W.L
Tidal zone
M.L.W.L
Concentrated
Corrosion
Submerged
Zone
Sea Bottom
Surface
Mud Zone
Depth

Figure 8.3 Representative figure for rate of corrosion in Vertical-direction of long steel members

In the splash zone, the structure is constantly subjected to seawater splashing, and therefore a large amount of seawater and oxygen can be supplied to steel surface. In general, mean corrosion rate in this zone reaches the level of **0.08 mm/year**.

8.5.2.1 Below bed level

Where piles are below the bed level in temperate and tropical locations, very little corrosion occurs due to low oxygen supply. *The corrosion rate given for the underground condition is applicable as 0.012 mm/year/side.*

8.5.2.2 Sea water immersion zone

Corrosion in the submerged zone is nearly uniform. *In temperate and tropical locations, the recommended mean corrosion rate at depths below –1 m level is about 0.035 mm/year/side.*

8.5.2.3 Tidal zone

The tidal zone is where structures undergo periodic immersion in seawater and exposure to the atmosphere due to tidal action. In this zone, corrosion rate in the vicinity of the mean seawater level (M.S.L.) is small; however, corrosion rate in the vicinity just beneath the M.L.W.L. is extremely large. In temperate locations, the recommended mean corrosion rate for design is in the order of 0.035 mm/year/side. *In tropical locations, the recommended mean corrosion rate for design is 0.04 mm/year/side*.

8.5.2.4 Splash zone

In the splash zone, the structure is constantly subjected to seawater splashing, and therefore a large amount of seawater and oxygen can be supplied to steel surface. In this zone, thick stratified rust layers can develop and at thicknesses above about 10 mm these tend to spall from the steel especially on curved parts such as the shoulders and the clutches of sheet piles. Removal of the corrosion product exposes fresh steel substrate and corrosion proceeds at a much higher rate than where the corrosion product remains on the surface. The boundary between splash and atmospheric zones is not well defined but corrosion rates reduce rapidly with distance above wave height. In temperate locations, the recommended mean corrosion rate for design is in the order of 0.075 mm/year/side.

8.6 Corrosion Loss for Steel in Temperate Locations

Traditionally, corrosion of steel piling in different environments is expressed in terms of a thickness loss per year per face, which implies that the corrosion rate throughout the life of a structure is uniform. Research has indicated that the gradual development of corrosion products on the surface of the steel tends to inhibit the corrosion process leading to a gradual reduction in the corrosion rate with time. Table 8.1 gives recommended thickness losses for steel in the specified environments for the given life of structure from EN 1993-5. These figures relate to temperate locations and take into account the corrosion rate reduction with time.

Table- 8.1: Loss of thickness (mm) per face due to corrosion of bearing and sheet piles in soils, with or without groundwater

Environmental Conditions		Life of structure (years)				
		5	**20**	**50**	**75**	**100**
Fresh water (river, ship canal)	Common (in the zone of high-water attack [water line])	0.15	0.55	0.90	1.15	1.40
	Very polluted (in the zone of high attack [water line])	0.30	1.30	2.30	3.30	4.30
Sea water	In the zone of high attack (Low water and splash zones)	0.55	1.90	3.75	5.60	7.50
	In the zone of permanent immersion or in the intertidal zone	0.25	0.90	1.75	2.60	3.50
Natural soils	Undisturbed (Sand, silt, clay schist)	0.00	0.30	0.60	0.90	1.20
	Polluted industrial sites	0.15	0.75	1.50	2.25	3.00
	Aggressive (Swamp, marsh, peat)	0.20	1.00	1.75	2.50	3.25
Non aggressive fills (clay, schist, sand)	Compacted	0.09	0.35	0.60	0.85	1.10
	Non-compacted	0.18	0.70	1.20	1.70	2.20
Aggressive fills (ashes, slag)	Compacted	0.25	1.00	1.63	2.25	2.88
	Non-compacted	0.50	2.00	3.25	4.50	5.75
Note: The corrosion allowances are in mm thickness loss per face for the given life of the structure						

8.7 Corrosion in Roadside Environments

For atmospherically exposed surfaces of piles in a roadside environment, it is recommended that the assumed loss of thickness over 125 years is 2 mm. However, where design assumptions are made regarding the nature of existing or re-used fill, these should be validated by testing.

8.7.1 Corrosion due to Pollution

Both saltwater and freshwater are prone to pollution. Water can be polluted with industrial water, sewage, spilled oil, and more. Some pollutants increase the oxygen level in the water by killing organisms that depend on it; others alter the pH balance of the water. Part of the reason soil may have a higher pH balance is contamination. Pollution, vegetation, and various chemicals can affect soil in different ways, which can have an effect on its interaction with steel (or other things, like water).

8.8 Corrosion Protection Systems

In many circumstances steel corrosion rates are low and the use of protective systems may be unnecessary. However, where corrosion of steel piling is more significant, methods of increasing the effective life of a structure may need consideration. The measures that can be taken include the following:

a) Sacrificial steel
b) Higher strength steel
c) Corrosion resistant micro-alloyed steel (AMLoCor)
d) Applying a protective organic coating
e) Cathodic protection
f) Concrete encasement

8.8.1 Sacrificial Steel

Engineering calculations will determine the critical performance criteria for a given steel member. The assumed section properties of the member must be present throughout the life of the structure for the factor of safety against failure stipulated in the design.

During design, the structural properties of sheet pile element (second moment of area, section modulus and cross-section area) will be known. However, the pile section chosen must allow extra sacrificial thickness due to corrosion loss. Based on the anticipated thickness loss, which may be derived by multiplying the corrosion rate by the life of the structure, reduced section properties for the section can be determined. These reduced section properties must satisfy the minimum requirements for design verifications.

The effective life of a steel pile member can be increased using an additional steel thickness as a corrosion allowance. It is important to consider the stress distribution in the structure in order to locate the region where corrosion losses are most critical. It is possible that the zones subject to the highest corrosion rates will not coincide with the most highly stressed zone and therefore the use of a corrosion allowance can be a cost-effective method of increasing effective life. Alternatively, adding extra pile thickness locally to compensate the corrosion loss may prove more economical.

When assessing pile durability, it is often beneficial to assess the pile drivability at an early stage where hard driving is expected (refer Chapter-11). In few cases, the steel section required for pile installation may be larger than that needed for structural requirement and the additional steel provided to accommodate the driving stresses can be utilised as sacrificial thickness of member.

There is a trend towards adding a sacrificial thickness of steel rather than painting piles because of the potential damage to the paint coating during installation. The design life of steel can usually be achieved by adding a sacrificial thickness of steel in submerged zone and other protection measures such as concrete encasement in intertidal zone.

8.8.2 Higher Strength Steel

For a chosen section, the effective design life of a pile can be increased by adopting steel with higher yield strength. It is important to note that, all grades of carbon steel have similar corrosion rates. Therefore a greater performance can be achieved by specifying the higher-grade steel.

However, care is required when considering higher-grade steel and a corresponding smaller thickness. A reduction in the second moment of area for the thinner section will result in a greater deflection of the pile or local buckling, which may not then be acceptable.

8.8.3 Corrosion Resistant Micro-Alloyed Steel (AMLoCor)

AMLoCor is a micro-alloyed steel that offers a significant reduction of the corrosion rates in the low water zone (LWZ) and in the permanent immersion zone (PIZ), in a marine environment compared to conventional sheet pile steels. This is new grade steel, very effective in PIZ and yield the most cost-effective solution in the long-term, without any additional steel protection. In-situ tests during the last two decades in Northern European ports have proven that, in comparison to standard structural steel in the critical zones, the loss of steel thickness of AMLoCor is reduced by a factor 3 in the PIZ and a factor 5 in the LWZ as depicted in **Figure 8.4**.

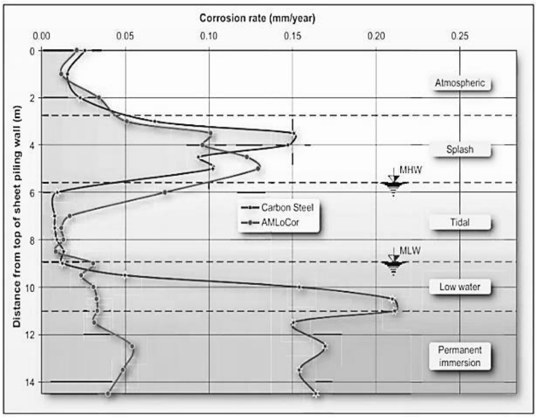


Figure 8.4 Measured corrosion rates in an European port

In many cases, the maximum bending moment, and consequently the steel stress, is often in an area where corrosion rates are relatively low, such as the permanent immersion zone or embedded zone. However, the Low Water Zone sometimes governs the design because of the higher rate of thickness loss in this area.

If AMLoCor is used, the reduced loss of thickness due to corrosion can be taken into account in the structural design verification. The loss of thickness data for conventional sheet pile steel grades in accordance with EN 10248 is presented in EN 1993-5. To obtain the loss of thickness using AMLoCor steel, the EN 1993-5 loss of thickness values are divided by a Corrosion Impediment Ratio (CIR), given in Table 8.3.

Table- 8.3: Corrosion Impediment Ratio (CIR) of Micro-Alloyed Steel

Zone	Low water	Permanent immersion	Splash
CIR	5	3	1

As the mechanical properties of AMLoCor are fully equivalent to conventional sheet piling steels, design structural resistances can be determined according to all relevant design codes for steel sheet piling structures such as EN 1993-5. Recent work has also shown better corrosion resistance of AMLoCor compared with conventional steel in soils. After a year of testing, a 20% decrease of the corrosion rate in silt loam soil was demonstrated. AMLoCor also performed better than steel protected with a hot dip galvanized layer in the same exposure conditions.

8.8.4 Protective organic coating

For some projects, additional corrosion protection is required to attain the desired product performance. Organic coatings offer a cost-effective approach to improved corrosion protection.

Coatings that are shop applied under controlled conditions are often more durable than site applied coatings. Since coatings can be damaged during transport, handling and pitching, appropriate on-site remedial treatment may be required. Driving in certain types of soil, e.g. gravels, may cause removal of some types of coatings, and this aspect should be considered when selecting an appropriate product.

The performance of a coating is affected by the quality of surface preparation prior to painting, the paint type and the thickness of coat applied. Paint manufacturers will generally give an indication in their product literature of the life to be expected from a given paint thickness. As a guide, a coating design life of the order of 15 - 20 years can be expected from an epoxy coating with a dry film thickness of 400 microns. A paint coating can be applied over part of piles such as the upper sections of the outer face only.

For anti-corrosive protection, paintings of solventless Polyethene (PP) or Polypropylene (PP) coat material with Epoxy zinc phosphate primer Coating can be applied on outer surface of exposed steel. Generally, 2.5mm to 3.0mm thick coatings are effective. It can be effectively applied at site by spraying or with brush.

8.8.5 Cathodic protection

The design and application of cathodic protection (CP) systems to marine piled structures is a complex operation requiring specialist knowledge and experience. The design needs to provide protection to the structure and prevent detrimental effects (such as damage to high strength steels), both to the protected structure and any third-party structures. It is recommended that specialist advice is sought when considering the use of cathodic protection for corrosion protection. The principals involved are outlined below.

Two systems are employed, utilising either sacrificial anodes or impressed DC currents. In normal electrochemical corrosion, all metal loss occurs at the anode and both types of CP system prevent corrosion by rendering the steel structure cathodic to externally placed anodes. Bare steel structures initially require an average current density of about 100 mA/m^2 in seawater, but this value normally falls over a long period of continuous operation to within the range 30 to 70 mA/m^2. Therefore, for a sheet piled structure of large surface area, the total current required could be considerable.

In many cases CP is used in conjunction with compliant coatings as indicated in **Figure 8.5**, hence if piles are coated below the water level then, depending upon the type of coating employed, current requirements are considerably reduced and can be as low as 5 mA/m^2.

Deterioration of the protective coating occurs with time, though this is counteracted to some extent by the deposition of protective calcium and magnesium salts on bare areas of the sheet piling and the growth of marine organisms.

However, in the long term, an increase in total current may be necessary and the CP system should be designed with an appropriate margin of capacity to cover this situation. This is explained in relevant design codes such DNV GL RP B401 which has breakdown factors for coatings and how that influences current density requirements. Not all protective coatings can be used in conjunction with CP. The coating should be of high electrical resistance, as continuous as possible, and resistant to any alkali which is generated by the cathodic reaction on the steel surface. The coating system suggested for sea water immersion can be used with CP.

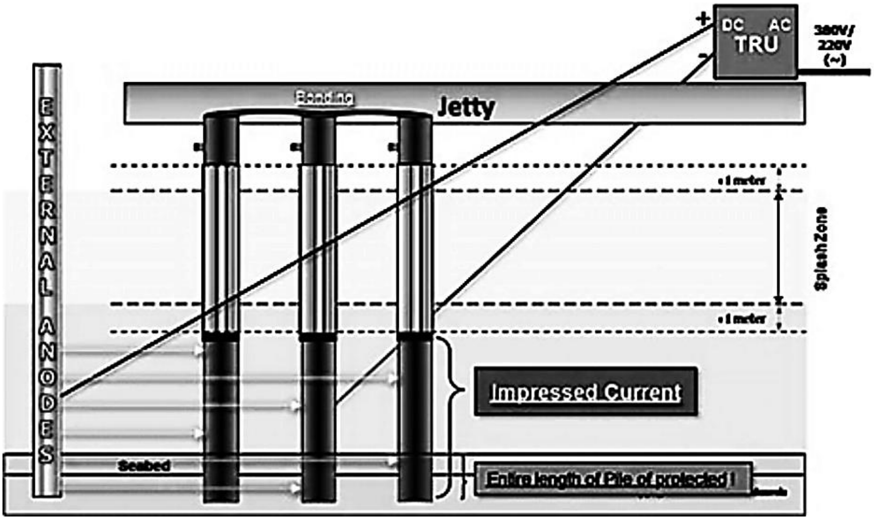


Figure 8.5 Representative Sketch of Cathodic Protection System

When considering CP it should be borne in mind that this method is considered to be fully effective only up to the half-tide mark. For zones above this level, including the splash zone, alternative methods of protection are required. It is considered that CP is effective at sea bed level where localised corrosion occurs due to sand eroding away the corrosion product layer (rather than the uncorroded steel surface). However, sand erosion prevents the deposition of protective calcareous deposits normally formed during CP and, therefore, the protective current density would be higher than typical values.

8.8.6 Concrete encasement

Concrete encasement can be used to protect steel piles in marine environments. Often the use of concrete is restricted to the splash zone by extending the concrete cope to below the mean high water level. However, in some circumstances, both splash and tidal zones are protected by extending the cope to below the lowest low water level which also gives protection against Accelerated Low Water Corrosion (ALWC) as indicated in **Figure 8-6**.

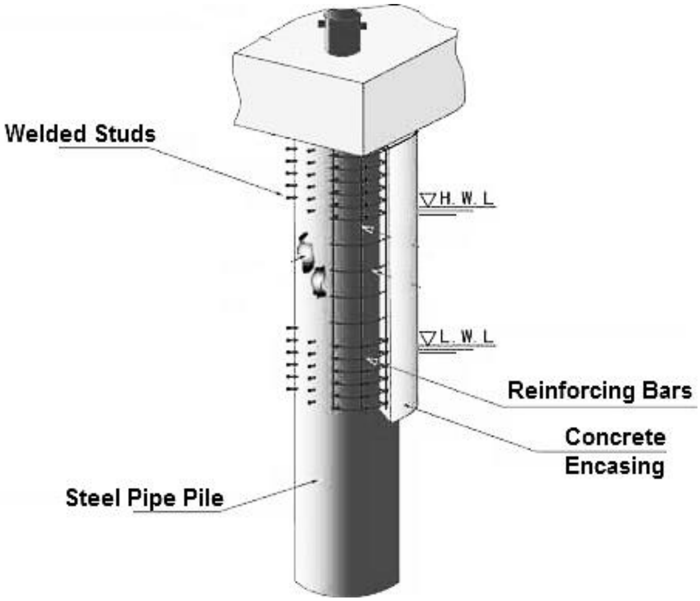


Figure 8.6 Representative Sketch of Concrete Encasement of Steel Pile

Experience has shown that where the splash zone is only partially encased, a narrow zone of increased corrosion can occur at the steel-concrete junction. This is a result of electrochemical effects at the steel-concrete junction, i.e. a potential difference is generated between steel in concrete and in seawater which, combined with the effects of differential aeration at the junction, causes the exposed steel immediately adjacent to the concrete to become anodic and corrode preferentially. Concrete is not itself always free from deterioration problems. It normally has a pH value of about 12 to 13 and within this pH range steel remains passive and corrosion is superficial. However, diffusion of chloride ions into the concrete from seawater can break down steel passivity and stimulate the corrosion reactions.

Therefore, concrete for protecting steel in seawater must be of good quality, i.e. have high strength, good bonding characteristics, and low permeability and be free initially from chlorides. It must also provide adequate cover and be properly placed and cured. If these requirements are not met, then rust formed from corrosion of the steel piles or steel reinforcement within the concrete can exert sufficient pressure to spall the concrete and expose the steel. In this case, the piles would need to be designed with a corrosion allowance. The exposure classification of the concrete should be determined according to EN 206.

8.9 Inspection and repair

Since the rate of corrosion loss depends on many parameters, it is prudent to undertake regular inspections to control and validate the residual thickness and the mechanical properties of a piled installation. This is particularly important for port and harbour structures. Inspection can be carried out in the dry using a boat or hydraulic access platform or an underwater survey may be carried out using divers or an underwater camera.

Chapter 9

Structural Design of Sheet Pile Wall

Engineering design is an activity other than purely manual and physical work which brings about the utilization of the materials and laws of nature for the good of works.

- *Anonymous*

9.1 General

After discussing the various methodology of structural analysis in chapter 6, the basis for design of pile or wall is discussed here. In fact, structural analysis is an integral part of structural design and structural design is the methodical investigation of the stability, strength and rigidity of structures. The primary purpose of a structure is to transmit or support the intended loads. Therefore, the design process of driven pile system involves a set calculation to identify the response of the structure against the design loads that will be required to resist during its lifetime. In this process,

- We determine the suitable geometry of the structure to achieve equilibrium under the design conditions verifying the requirements of wall for all applicable loads, bending moments and shear forces.
- We identify the types of effective sections, combinations, material grades, layout and dimensions of various components suitable for a particular purpose and economy of material.
- We verify, whether the selected section would be able to bear the different combination of loads without failure, i.e. collapse, damage, and even deflection, or vibration.
- And finally, based on sectional requirements installation and execution process is finalised.

Generally, the design starts with conceptualisation of structure. In this phase, the consideration of various requirements such as layout of structure, feasible dimensional details, availability of space for construction etc. determine the choice of one or perhaps several alternatives for structure. The alternative which offers the best general and economical solution is verified. The basic necessity of the designed structural elements is to meet the internal functional requirements safely and effectively.

9.2 Design Concepts

9.2.1 Working Stress Method

The traditional method for design of steel structure is allowable stress design (ASD) or elastic design method. It is based on allowable stress intensities, chosen in accordance with the concept that stress or strain correspond to the yield point of the material should not be exceeded at the most highly stressed points of the structure. The objective of this method is to ensure that a structure is capable of supporting the applied working loads safely. Working loads are service loads supposed to be applied on the structure. However, the disadvantage of allowable stress design method is that, it does not provide a uniform overload capacity for all parts and all types of structures.

9.2.2 Limit State Design Method

The design philosophy adopted here for structural design is based on limit state principles, in which a distinction is made between ultimate and serviceability limit states. **Ultimate limit states** are concerned with the safety of people and the structure. Examples of ultimate limit states include loss of equilibrium, excessive deformation, rupture, loss of stability, transformation of the structure into a mechanism, and fatigue.

Serviceability limit states are concerned with the functioning of the structure under normal use, the comfort of people, and the appearance of the structure and construction works.

In this method, the working loads are factored before being applied for structural verification. The load factors are generally based on relevant code of practice and applicable standards for the project. The load factors are determined by probabilistic theory which account for

- Variability of anticipated loads
- Errors in design methods and computations
- Lack of understanding of material behaviour

9.3 Design of Pilings Based on Euro Codes

In present section, the structural design recommendations are based on Euro codes, particularly for steel piles and sheet pile walls. The general requirements for the limit state design of steel structures are covered by Eurocode3.

Eurocode3: Part I describes the general rules governing the design of steel structures. Eurocode3: Part 5 - Piling, outlines the limit state principles relating to the structural design of bearing piles, sheet piles, anchors, waling beams and bracing elements made from steel.

It has following salient features and design guidelines;

- These Eurocodes allow the use of elastic and plastic design for steel structures which, in turn, may allow for more economic use of materials, i. e. for appropriate sections.
- Based on physical testing of sheet pile sections and numerical investigations, the effect of wall plasticity and oblique bending for lack of shear force transmission in the interlocks are introduced for real design situations. This allows reduced bending stiffness and section capacity of U Piles.
- Other factors to be assessed include the possible reduction in strength of single and double U-piles (oblique bending) and Z-piles, due to local effects of differential water pressure greater than 5 m.
- European Standard on shell design provides the designer a comprehensive and very general tool to assess the thin-walled tube sections. The design of (cylindrical) shell structures regulated in EN 1993 part 1-6 (CEN 2010), deal with the design of tubular

combi-wall which offers a very promising solution for design of contiguous type structure in harbour construction.

- The structural design of sheet pile to Eurocode involves different sets of calculations to determine the structural requirements of the wall to resist bending moments, shear forces and axial loads taking care of;
  - choice of suitable materials;
  - appropriate design and detailing;
  - Procedures for execution and use of the structure, which will be discussed in detail in preceding chapters.

Verification of limit state structure involves checking for actions and the effect of loads which do not exceed their corresponding design resistances. This is expressed by the inequality:

$\mathrm{E_d} \leq \mathrm{R_d}$ with

$\mathrm{E_d}$ design effects of actions;

$\mathrm{R_d}$ the corresponding design resistance.

The first impression when reading the standard EC 3-5 is that this design approach is much more complex than ASD, probably because it considers the interaction between bending moments, shear forces and compression loads, as well as buckling length when applicable, and so on. However, in most cases, the governing factor is the bending moment M_{Ed} because the interaction between the effects can be neglected in most cases, or does not have a significant influence on the result. EC 3-5 considers both plastic as well as elastic sections modulus (W_{pl} or W_{el}), which depend on the classification of the section.

9.3.1 Structural Resistance of Steel Sheet Piles

In 1953 Brinch Hansen developed a design method with plastic hinges for sheet pile walls and ever since this method has formed the basis for the current Danish design practice. Later Windels and Weißenbach developed verification methods for plastic design of steel sheet pile walls were applied to quay walls in the port area of Hamburg, which generates yielding of the outermost fibre as:

$$M_{Sd} \leq f_y \, W_{el} / \gamma_M \qquad (9.1)$$

Which is the basis for Eurocodes and further the selection of a sheet pile section shall take into account the choice of the steel grade. It has an influence on the classification of the sections, so that it is not possible to give a simple rule covering all the cases, but generally speaking, using a higher steel grade (higher yield strength) leads to a lighter sheet pile, hence to a more cost-effective solution. Note that the additional premium for a higher steel grade is by far offset by the weight reduction of the sheet pile.

9.3.2 Classification of Cross-Sections

"Eurocode3" classifies sections into four different classes, which identify the extent to which the resistance and rotation capacity is limited by local buckling. The rotation capacity is dependent on the geometrical stability of the cross-section of the sheet pile, and in particular on the slenderness and steel grade of the compression flange and web.

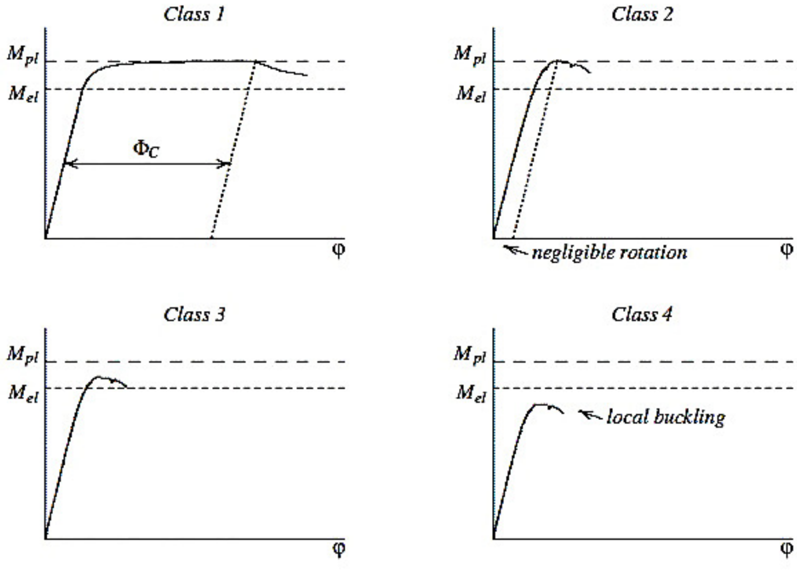


Figure 9.1 Classification of Cross-sections

As these parameters differ for every sheet pile profile, the cross-sectional behaviour of the sheet pile can be subdivided into four classes which are in accordance with Eurocode3, part 1-1 as shown in **Figure 9.1**:

- Class 1 – section can form a plastic hinge with the rotation capacity required from plastic analysis, without reduction of the resistance.
- Class 2 – section can develop its plastic moment capacity, but has limited rotation capacity.
- Class 3 – section in which the stress in the extreme compression fibre of the section, assuming an elastic distribution of stresses, can reach the yield strength, but local buckling is likely to prevent the development of the plastic moment capacity.
- Class 4 – section is subject to local buckling before reaching the yield stress in one or more of the parts.

9.3.3 Determination of Rotation Capacity

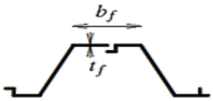
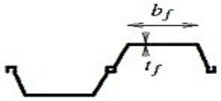
9.3.3.1 Z and U Profile Sheet Piles

According to Hartmann-Linden the rotation capacity of steel sheet piles in bending can be described by the slenderness and the steel grade of the compression flange, using the b_f / t_f ratio and ε, where b_f and t_f are the width and thickness of the compression flange and ε is a function of the steel grade, defined by

$$\varepsilon = \sqrt{\frac{235}{f_y}} \qquad \text{with } f_y \text{ in } N/mm^2 \qquad (9.2)$$

Differentiation must be made between U and Z-profiles when determining their moment resistance and rotation capacity of pile in Table - 9.1.

Table 9.1: Design rules for classification of cross-sections (according to ENV 1993-5)

<table>
<tr><td colspan="2"></td><td colspan="3">Z - Profile</td><td colspan="2">U - Profile</td></tr>
<tr><td colspan="2">$\varepsilon = \sqrt{\frac{235}{f_y}}$; f_y in N/mm^2</td><td colspan="3"> </td><td colspan="2"> </td></tr>
<tr><td colspan="2">Class -1</td><td colspan="5">a rotation check has to be carried out</td></tr>
<tr><td colspan="2">Class -2</td><td colspan="3">$b_f/t_f \leq 45\,\varepsilon$</td><td colspan="2">$b_f/t_f \leq 37\,\varepsilon$</td></tr>
<tr><td colspan="2">Class -3</td><td colspan="3">$b_f/t_f \leq 66\,\varepsilon$</td><td colspan="2">$b_f/t_f \leq 49\,\varepsilon$</td></tr>
<tr><td>f_y (N/mm^2)</td><td>240</td><td>270</td><td>320</td><td>355</td><td>390</td><td>430</td></tr>
<tr><td>ε</td><td>0.99</td><td>0.93</td><td>0.86</td><td>0.81</td><td>0.78</td><td>0.74</td></tr>
</table>

The interlock of a Z-profile has a stiffening effect on the buckling resistance of the compression flange, which leads to a higher moment resistance. U-sections, on the other hand, have a higher rotation capacity than Z-sections for a low slenderness ratio, because the web of U-sections is stiffened by the interlock, which favours the rotation capacity. The boundaries for b_f/t_f ratio and ε between the different cross-sectional classes of Z-profiles and U-profiles given by ENV 1993-5 are presented in Table 9.1. Similarly, for H-piles, HAT piles and tubular piles are indicated in Table-9.2, 9.3 and 9.4 in accordance with ENV 1993-1-1. The rules for U-sections are derived from sections with welded interlocks but the formulae are conservative for cases with a limited interlock friction capacity as discussed in section 9.3.4. The b_f/t_f and ε ratio is related to a maximum moment level of $M_{max} = M_{pl} = f_y W_{pl}$.

9.3.3.2 H-Piles and Flanged Sections

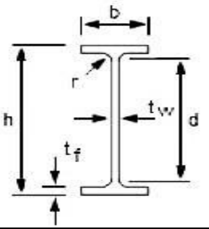
Sections with slender webs or flanges are more susceptible to local buckling, where the element will fail before the design strength is reached. Eurocode3 takes into account the effects of local through the process of cross section classification.

The limits between the classes depend on the ε factor which is calculated using fy, the yield strength of the steel. The limits of material yield strength, fy and c/t ratio for flanged sections are given in table 9.2;

Table 9.2: Cross-section Classification (flanged Section)

	Class 1	**Class 2**	**Class 3**
Flange Outstand	$c/t_f \leq 9\,\varepsilon$	$c/t_f \leq 10\,\varepsilon$	$c/t_f \leq 14\,\varepsilon$
Web in bending	$d/t_f \leq 72\,\varepsilon$	$d/t_f \leq 83\,\varepsilon$	$d/t_f \leq 142\varepsilon$
Web in compression	$d/t_f \leq 33\,\varepsilon$	$d/t_f \leq 38\,\varepsilon$	$d/t_f \leq 42\,\varepsilon$

Table 9.3: Definitions of flange outstand and web compression depth

Flange Outstand (c)	$c = (b - t_w - 2r)/2$	
Internal Compression Part (d)	$d = h - 2t_f - 2r$	

9.3.3.3 Tubular (Pipe) Pile Sections

Table 9.6: Cross-section Classification for Tubular Piles and other sections based on [*EN 1993-1-1*], the limiting values are d/t ratio. The limits of yield strength, fy and d/t ratio for tubular sections are given in Table 9.4.

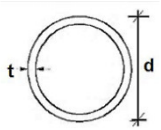


Table 9.4: Cross-section Classification (tubular section)

Shape	Class 1	Class 2	Class 3
Tubes / Pipes	$d\,/\,t \leq 50\varepsilon^2$	$d\,/\,t \leq 70\varepsilon^2$	$d\,/\,t \leq 90\varepsilon^2$

Design of tubular pile wall with $d\,/\,t \geq 90\varepsilon^2$, are given section-9.7.

An element that doesn't meet the class 3 limits should be taken as a class 4 section. Effective widths are assigned to Class 4 compression elements to make allowance for the reduction in resistance as a result of local buckling. To calculate the effective width of a Class 4 section, refer to the relevant section in the following Eurocodes:

Section Type	Reference
Cold-formed sections	EN 1993-1-3
Hot-rolled and fabricated section	EN 1993-1-5
Circular Hollow Sections (Pipes)	EN 1993-1-6

9.3.4 Oblique Bending in Sheet Piles

In steel sheet pile walls built up of U-sections the interlocks of the individual U-piles are on the central bending axis of the sheet pile wall, see **Figure 9.2**. If sufficient shear friction capacity of the interlocks is present, the central bending axis is the neutral axis and the maximum cross-sectional resistance is obtained. Sheet pile producers often provide the cross-sectional data of a U-section for the maximum cross-sectional resistance. For design of structure, UK National Annex or Technical Advisory Notes by sheet piling group (www.steelpilinggroup.org) may be referred.

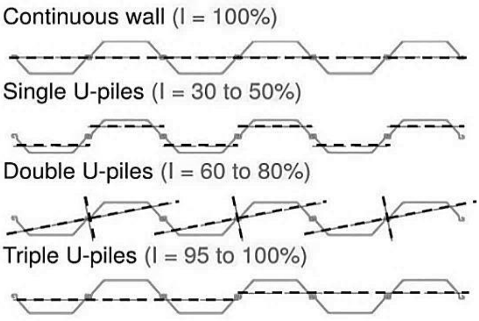

Continuous wall (I = 100%)
Single U-piles (I = 30 to 50%)
Double U-piles (I = 60 to 80%)
Triple U-piles (I = 95 to 100%)

Figure 9.2 Oblique Bending in U-Sheet pile Sections

In European practice, sheet pile walls composed of U-sections are generally built either with single piles, where the interlocks are threaded together during pile installation, or with double piles, in which two piles are threaded together and crimped or welded before pile installation, and occasionally use is made of triple piles .

The shear force transmission in the interlocks may not always be sufficient to assume the maximum cross-sectional resistance. In extreme cases, if the shear capacity of the interlocks is absent, single, double, and triple U-piles behave as explained below:

- Single U-piles bend about the principal axis of inertia of an individual pile. This causes a reduction of the moment of inertia to approximately 30% and a reduction of the section modulus to 50% of the maximum cross-sectional resistance values
- Double U-piles bend about the inclined principal axis of inertia, which causes a deflection both perpendiculars to the wall and in plane of the wall. The moment of inertia reduces to approximately 60% and the section modulus to about 80% of the maximum cross-sectional resistance values
- Triple U-piles bend about the principal axis which causes a reduction to approximately 95% for the moment of inertia and to about 100% for the section modulus of the maximum cross-sectional resistance values.

9.3.5 Effective length of pile

Buckling length of pile may be estimated as; $L_{crit} = kH$;

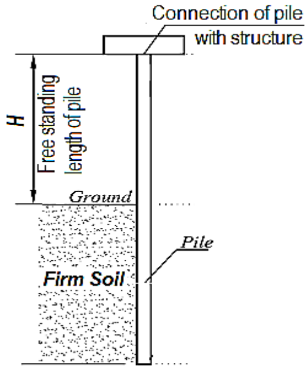


Figure 9.3 Estimation of buckling length for bearing piles

The value for 'k' is based on connection between the pile and concrete deck or capping structure. Based on BS EN 1993-5, the values of 'K' for estimation of buckling length are given in table-9.5.

Table - 9.5: Coefficient for fixidity length of pile

Condition of pile connection with concrete deck	k
Translational fixed and rotational free	1.0
Translational fixed and rotational fixed	0.7
Translational free and rotational fixed	2.0

9.3.6 Local Effects of Water Pressure

According to the BS EN 1993-5, in the case of differential water pressure exceeding 5 m head for Z-piles and 20 m head for U-piles the effects of water pressure on transverse local plate bending should be taken into account to determine the overall bending resistance. As a simplification, this verification may be carried out for Z-piles using the following procedure:

- if the differential water pressure is more than 5 m head, the cross-sectional verification should be carried out at the locations of the maximum overall bending moments;

- the effect of differential water pressure should be taken into account by using a reduced yield strength;

$$f_{y,red} = \rho_p f_y \tag{9.3}$$

The values of ρ_p as reduction factors for differential water pressure are provided in Eurocode3 as given in Table: 9-6.

Table 9-6: Reduction factors ρ_p for Z-piles due to differential water pressure or head (w)

Water head, (m)	Values of reduction factor ρ_p for Z-piles			
	$(\boldsymbol{b}/\boldsymbol{t}_{min})$, ε =20	$(\boldsymbol{b}/\boldsymbol{t}_{min})$, ε =30	$(\boldsymbol{b}/\boldsymbol{t}_{min})$, ε = 50	$(\boldsymbol{b}/\boldsymbol{t}_{min})$, ε = 50
5.0	1.00	1.00	1.00	1.00
10.0	0.99	0.97	0.95	0.87
15.0	0.98	0.96	0.92	0.76
20.0	0.98	0.94	0.88	0.60

Where, b is the width of the flange, but b should not be taken as less than $c/\sqrt{2}$, where c is the slant height of the web;

t_{min} is lesser of the t_f or t_w

t_f is the flange thickness

t_w *is* the web thickness

w is the differential water head in m

$\varepsilon = \sqrt{\frac{235}{f_y}}$; f_y is the yield N/mm^2

9.4 Verification of Sections

The Table 9.2 identify the limiting width-to-thickness ratios for classifying the various parts of the flanged-section. The parameters used in calculating the width-to-thickness ratio limits are defined below in Table 9.7:

Table 9.7: Limiting width to thickness for classification of cross-sections

Shape	Part	Ratio	Class 1	Class 2	Class 3
Tubes / Pipes	Wall	d/t	$50\varepsilon^2$	$70\varepsilon^2$	$90\varepsilon^2$
Boxes	Web, flange	c/t	72ε	83ε	124ε
General High Modulus Sections, HAT piles, H-Z Combi-Piles	Section	N/A	Assumed as Class 3		

9.4.1 Design of Z and U Profile Sheet Piles

The moment check for each section should satisfy the followings;

9.4.1.1 Moment Check

$$\frac{M_{Ed}}{M_{c,Rd}} \leq 1.0 \tag{9.4}$$

M_{Ed} is design bending moment,

$M_{c,Rd}$ is the design moment resistance of the cross-section

$M_{c,Rd} = \beta_B\ W_{pl}.f_y/\gamma_{Mo}$ (for class 1 or 2 sheet pile sections)

$= \beta_B W_{el} f_y/\gamma_{M0}$ (for class 3 sheet pile sections)

β_B = 1.0 for Z-piles, triple U-piles and continuous U-pile wall

$\beta_B \leq 1.0$ for single and double U-piles (Refer - National Annex Reduction Factors for U shaped sheet piles to EC3-Part 5)

The plastic and elastic section modulus values, W_{pl} and $W_{el,}$ are part of the sheet pile sections definition. The effect of high shear on the design moment resistance, $M_{c,Rd}$ is considered if:

$$V_{Ed} \geq 0.5 V_{pl},R_d \tag{9.5}$$

To account for the effect of high shear in wall subjected to major axis moment, the reduced design plastic resistance moment is taken as:

$$M_{V,Rd} = \left[\beta_B W_{pl} - \frac{\rho {A_V}^2}{4 t_w \sin\alpha}\right]\frac{f_y}{\gamma_{M0}} \leq M_{c,Rd} \tag{9.6}$$

$M_{V,Rd}$ is the reduced plastic design moment.

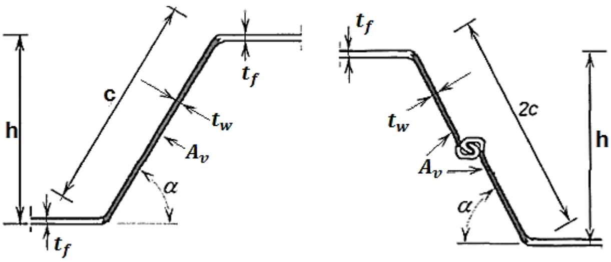
$$\rho = \left(2\, V_{Ed}/V_{pl,Rd} - 1\right)^2 \tag{9.7}$$

Value of A_V shall be adopted from equation 9.9.

9.4.1.2 Shear Check

The shear check for each section should satisfy the followings;

$$V_{Ed}/V_{pl,Rd} \leq 1.0 \tag{9.8}$$



$$c = \frac{h - t_f}{\sin\alpha} \qquad\qquad c = \frac{h - t_f}{2.\sin\alpha}$$

a) Z - Pile b) U - Pile

Figure 9.4 Definition of Shear area for sheet Piles

$V_{pl,Rd}$ $=\frac{A_V f_y}{\sqrt{3}\gamma_{M0}}$; the design plastic shear resistance for each web of Z and U-profile section as shown in **Figure 9.4**.

Where, A_V is the projected shear area for each web, *h* is the overall height, t_f is the flange thickness and t_w is the web thickness of sheet pile section.

$$A_V = t_w(h - t_f) \tag{9.9}$$

$$V_{pl,Rd} = \frac{A_V f_y}{\sqrt{3}\gamma_{M0}} \tag{9.10}$$

9.4.1.3 Axial Buckling Check

For combined bending and compression, the axial buckling check is not ignored if:

$$\frac{N_{Ed}}{N_{cr}} \leq 0.04 \tag{9.11}$$

N_{Ed} is the design axial compressive force,

N_{cr} is the elastic critical load of sheet pile, which may be adopted as

$$N_{cr} = EI\,\beta_D\pi^2/l^2 \tag{9.12}$$

β_D a factor generally equal to 1

l effective length of pile

9.4.2 Other factors for Design of Sheet Pile Cross Section

i) The values of the effects of actions should not exceed the design resistance of the cross-section.

ii) The design resistance should be determined taking into account a carefully chosen structural design model taking account for appropriate virtual geometric model of structure. If required the reduction of cross section properties due to a loss of thickness induced by corrosion should be accounted.

iii) For U-piles possible lack of shear force transmission in the interlocks should be taken into account.

iv) If the sheet piling is subject to transverse bending due to differential water pressure, the effects of the water pressure should be taken into account.

v) Elastic resistance may be used for class I and class 2 cross-sections.

If no moment redistribution, and therefore no plastic rotation, is taken into account for class 1 or 2 Profiles, determination of the effects of actions for the verification of the cross-section may be carried out using an elastic beam model.

Example : 9.1

Following U-type sheet pile section is used for temporary propped cantilever wall. The ultimate design bending moment, shear force and vertical compressive force on wall are 420 KN.m/m, 380 KN/m and 20 KN/m respectively. The section property of proposed U-type sheet pile is as given below;

Pile section width	*B*	***600***	*mm*
Length of flange	*b*	***269***	*mm*
Flange thickness	t_f	***10.2***	*mm*
Web thickness	t_w	***8.4***	*mm*
Overall height of section	*h*	***430***	*mm*
Web angle	α	***57.5***	*degrees*
Moment of Inertia	*I*	***35950***	cm^4/m
Plastic section modulus	W_{pl}	***1988***	cm^3/m
Elastic section modulus	W_{el}	***1670***	cm^3/m
Cross section	*A*	***154.2***	cm^2/m

Check the structural adequacy of sheet pile section for applicable loads.

Solution:

Design compressive force (N_{ED})	=	**20.0**	KN/m
Design bending moment (M_{Ed})	=	**420.00**	KN.m/m
Design shear force (V_{Ed})	=	**380.0**	KN/m
Pile Section	=	**U-type**	
Pile yield strength (f_y)	=	**390**	N/mm^2
Pile Young's Modulus (Es)	=	**210000**	N/mm^2
Partial material factor (γ_{M0})	=	**1.00**	
Design life	=	**0.00**	years

Pile section properties :

		Full properties		*Reduced properties (allowing for corrosion losses)*	
Pile type		**U**		U	
Pile section width	B	**600**	mm	**600**	mm
Length of flange	b	**269**	mm	**269**	mm
Flange thickness	t_f	**10.2**	mm	**10.2**	mm
Web thickness	t_w	**8.4**	mm	**8.4**	mm
Overall height of section	h	**430**	mm	**430**	mm
Web angle	α	**57.5**	degrees	**57.5**	degrees
Moment of Inertia	I	**35950**	cm^4/m	**35950**	cm^4/m
Plastic section modulus	W_{pl}	**1988**	cm^3/m	**1988**	cm^3/m
Elastic section modulus	W_{el}	**1670**	cm^3/m	**1670**	cm^3/m
Cross section	A	**154.2**	cm^2/m	**154.2**	cm^2/m
	EI	**75495**	$KN.m^2/ m$	**75495**	$KN.m^2/ m$
	β_B	0.75			
	β_D	0.45			

Design Verifications

i) Shear Capacity

Project shear area (A_v) = $t_w(h - t_f)$ = **58.77** cm^2/m

$V_{plRd} = A_v f_y / \sqrt{3}\ \gamma_{M0}$ = **1323.35** KN / m

Design shear force (V_{Ed}) = **380.0** KN / m (V_{Ed} < $V_{pl\,Rd}$)

Shear Capacity Satisfactory

$V_{Ed} / V_{pl\,Rd}$ = **0.29** **<** 1

ii) Web Shear Buckling Resistance

$c = 0.50\,(h - t_f)/\sin\alpha$ $\quad c = $ **248.9** mm

$\varepsilon = \sqrt{235/f_y}$ $\quad \varepsilon = $ **0.78**

$c/t_w = $ **29.63**

$72\varepsilon = $ **55.89**

for, $c/t_w < 72\,\varepsilon$ **Web Shear buckling need not verified**

$\bar{\lambda} = 0.346\,\frac{c}{t_w}\sqrt{\frac{f_y}{E}}$ = **N/A**

f_{bv} = **N/A** N/mm^2

$V_{b,Rd} = \dfrac{(h - t_f)\,t_w f_{bv}}{\gamma_{M0}}$ = **N/A** KN / m

iii) Cross-section Class

$e = \sqrt{(235/f_y)}$ $\quad \varepsilon = $ **0.78**

$(b/t_f)/\varepsilon = $ **33.97**

Cross-section class = **Class 1 & 2**

However, structure is designed, considering Class 3 or elastic section.

iv) Moment Resistance Capacity of Pile

Section modulus (W) = **1670** cm^3/m

Mc = $\beta_B W_{pl}\, {}^{f_y}/_{\gamma_{M0}}$ or $\beta_B W_{el}\, {}^{f_y}/_{\gamma_{M0}}$ = **488.48** KN.m / m

${}^{V_{Ed}}/_{V_{pl\,Rd}}$ = **0.29** < 1

$\rho = (2.V_{Ed}/V_{plRd} - 1)^2$ $\quad \rho$ = **N/A**

Reduced design moment (M_{CRd}) = **488.5** KN.m / m V_{Ed} < 50% capacity; No capacity reduction;

Design bending moment (M_{Ed}) = **420.00** KN.m / m

Moment capacity satisfactory

v) Combined Bending and Axial (Compression) Stress

Wall type **Propped**

Unsupported length (L) = **6.0** m

Effective buckling length factor = **1.0**

Effective buckling length (l) = **6.0** m

$N_{cr} = E\,I\,\beta_D\,\pi^2/\,l^2$ N_{cr} = **9313.82** KN / m

N_{Ed}/N_{cr} < 0.04 N_{ED}/N_{cr} = **0.0021** Buckling resistance does not need to be verified.

Buckling resistance check **N/A**

vi) Plastic design Resistance

$N_{pl\,Rd}$ = A f_y / γ_{M0} $N_{pl\,Rd}$ = **6013.8** KN / m

$N_{Ed}/N_{pl\,Rd}$ < 1.0 $N_{Ed}/N_{pl\,Rd}$ = **0.0033** Axial capacity satisfactory

Effect of axial force on plastic moment of resistance

Cross section class = **2**

Pile profile = **U**

Axial load to plastic capacity ratio limit = **0.00333** < **0.25**
No reduction in moment capacity required for U-piles

Moment reduction factor (F_N) = **1** Based on cross-section class and pile profile above

Reduced design moment (M_{CRd}) = **488.48** KN.m / m from above

$N_{Ed}/N_{pl\,Rd}$ = **3.3E-03**

$M_{N\,Rd} = F_N M_{CRd}\left(1 - N_{ED}\,/N_{pl\,Rd}\right)b$ = **488.48** KN.m / m

Design bending moment (M_{Ed}) = **420.00** KN.m / m

Moment capacity satisfactory

vii) Combined Bending, Axial and Shear Force Effects

V_{ED} = **380.0** KN / m

$V_{pl\,Rd}$ = **1323.35** KN / m

V_{Ed} /$V_{pl\,Rd}$ = **28.72** % As ratio is less than 50%; No capacity

$\rho = \left(2.V_{Ed}/V_{plRd} - 1\right)^2$ ρ = **N/A**

$f_{yRd} = (1 - r)\, f_y$ = **N/A**

$M_{CRd}\dfrac{f_{yRd}}{f_y}$ = **N/A** KN.m / m

$M_{CRd}\dfrac{f_{yRd}}{f_y}$ = **N/A** KN / m

$M_{N\,Rd} = F_N M_{CRd}\left(1 - N_{ED}/N_{pl\,Rd}\right)$b = **488.475** KN.m / m

Design bending moment (M_{Ed}) = **420.00** KN.m / m

Moment capacity satisfactory

9.5 Design of Tubular piles

Steel pipe piles consist of either rolled straight seam (LSAW) or spiral welded pipes (SSAW). Spiral weld pipe pile can be manufactured in various diameters with wall thicknesses up to 1 inch (25 mm) which is generally considered as most economical for fabricating pipes.

Uses of higher grade of steel allow higher stress in material to minimise the shell thickness. Therefore, thin walled tubular piles are designed as a shell structure using the characteristic of membrane stresses instead of bending stresses for transferring imposed loads. Their load carrying capacity under compression is either limited by a buckling failure (sudden change of the shape) in form of bifurcation or a limit point buckling (snap-through). The value of $D/t \geq 90\varepsilon^2$ is the limit for Class 4 cross-sections, however, it should be limited to 300 for piling applications as specified in API.

In Class-4 cross-sections, local buckling will occur before the attainment of yield stress in one or more parts of the cross-section, where the element fails before the design strength is reached. Eurocode3 [EC-3] takes into account the stress-based design approach in EN 1993-1-6 as a conservative approach, to take care of local buckling and other influences to assess the structural capacity of piles.

The structural adequacy according to the Eurocode needs an assessment with limit state buckling design of tubular piles involving the consideration of the following stress components:

a) axial membrane stresses from axial forces and bending moments with limit state buckling design against axial compression

b) shear stresses from transverse forces with limit state buckling design against shear

c) circumferential stresses from external pressure with limit state buckling design against circumferential compression

A brief detail of design procedure for tubular steel pile (as class-4 structure) is given below;

9.5.1 Design dimensional parameters

i) **Mean Dia. of pile**

The class-4 tubes are designed as shell and it should be represented by its middle surface or mean diameter of shell.

ii) **Buckling length of pile**

Buckling length of tubular piles should be estimated as explained in section-9.3.5. Appropriate boundary conditions are suggested for limit states verifications according to the standard conditions shown in Table - *9.8.* For special conditions, fixidity condition of pile needs to be assessed analytically.

Table 9.8: Boundary conditions for shells

Description of support	**Boundary condition code**
Transnationally and rotationally fixed	BC1
Pinned	BC2
Translational fixed and rotational free	
Free edge	BC3

9.5.2 Verification of Buckling

(Against Axial Compression and Bending)

The elastic axial buckling checks for tubular may be ignored if:

$$\frac{r}{t} \leq 0.03\,\frac{E}{f_y} \qquad (9.13)$$

Otherwise, the design check shall satisfy the following conditions in accordance EC-3.

$$\sigma_{x,Ed} \;\leq\; \sigma_{x,Rd} \qquad (9.14)$$

$\sigma_{x,Ed}$ is design axial stress,

$\sigma_{x,Rd}$ is design resistance of axial buckling,

$\sigma_{x1} = N/2.\pi.r.t$ N is the axial load for Limit State Design

$\sigma_{x2} = M\,/\,(\pi.r^2.t)$ M is the Moment for Limit State Design

$$\sigma_{x,Ed} = \sigma_{x1} + \sigma_{x2} \tag{9.15}$$

9.5.3 Computation of $\sigma_{x,Rd}$

i) The critical elastic axial buckling stress shall be evaluated using following expression;

$$\sigma_{x,Rcr} = 0.605\, E\, C_x \frac{t}{r} \tag{9.16}$$

Where,

t is the thickness of tube

r is the mean radius of cell

ii) Value of C_x shall be obtained from Table- 9.9.

Table 9.9: Classification of cylinder (tubular pile) based on dimensionless length parameter;

Description of cylinder	**Boundary conditions**	C_x
Long cylinder	$\omega > 0.5\, r/t$	$1 + \frac{0.2}{C_{xb}}\left[1 - 2\,\omega . {t}/{r}\right] \ngtr 0.6$
Medium length cylinder	$1.7 \leq \omega \leq 0.5\, r/t$	1.0
Short cylinder	$\omega < 1.7$	$1.36 - \frac{1.83}{\omega} + \frac{2.07}{\omega^2}$

Where, ω is the dimensionless length parameter characterises the length of the shell segment:

$$\omega = l/\sqrt{r.t} \tag{9.17}$$

Value of C_{xb}, the meridional buckling stress for long cylinders are given in Table- 9.10.

Table- 9.9: Value of C_{xb} for meridinal buckling stress in long cylinders

Case	Boundary condition for pile ends fixidity		C_{xb}
1	end 1 end 2	BC 1 BC 1	6
2	end 1 end 2	BC 1 BC 2	3
3	end 1 end 2	BC 2 BC 2	1

iii) Computation of relative shell slenderness parameter $\bar{\lambda}_x$ for axial stress

$$\bar{\lambda}_x = \sqrt{f_y / \sigma_{x,Rcr}} \tag{9.18}$$

iv) Computation of reduction factor χ_x due to axial buckling based on relative slenderness of the shell $\bar{\lambda}_x$ from the following inequalities;

$$\chi_x = 1 \qquad \text{when} \quad \bar{\lambda}_x \leq \bar{\lambda}_0 \tag{9.19}$$

$$\chi_x = 1 - \beta \left(\frac{\bar{\lambda} - \bar{\lambda}_0}{\bar{\lambda}_p - \bar{\lambda}_0} \right)^{\eta} \qquad \text{when } \bar{\lambda}_0 < \bar{\lambda}_x < \bar{\lambda}_p \tag{9.20}$$

$$\chi_x = \frac{\alpha_x}{\bar{\lambda}_x} \qquad \text{when} \quad \bar{\lambda}_p \leq \bar{\lambda}_x \tag{9.21}$$

Where:

α_x is the elastic imperfection reduction factor

β is the plastic range factor ; $\beta = 0.60$;

η is the interaction exponent ; $n = 1.0$

$\bar{\lambda}_0$ is the squash limit of relative slenderness; $\bar{\lambda}_0 = 0.20$

$\bar{\lambda}_p$ is the plastic limit of relative slenderness to be determined from expression 9.22.

$$\bar{\lambda}_p = \sqrt{\frac{\alpha_x}{1-\beta}} \tag{9.22}$$

The imperfection reduction factor α_x should be obtained from;

$$\alpha_x = \frac{0.62}{1+1.91(\Delta w_k/t)^{1.44}} \tag{9.23}$$

Where, Δw_k is parameter for imperfection amplitude:

$$\Delta w_k = \frac{1}{Q}\sqrt{\frac{r}{t}}.t \tag{9.24}$$

Where, Q is the fabrication quality parameter based on fabrication tolerance quality.

Table 9.11 : Values of fabrication quality Parameter Q

Fabrication tolerance quality class	Description	Q
Class A	Excellent	40
Class B	High	25
Class C	Normal	16

v) Computation of axial Buckling resistance $\sigma_{x,Rd}$

$$\sigma_{x,Rd} = \chi_x \cdot f_y/\gamma_{M1}; \tag{9.25}$$

Where, $\gamma_{M1} = 1.1$;

9.5.4 Verification of Buckling of tube against Shear Forces

The verification for buckling of tubes against shear forces can be carried out in accordance with provision and empirical relations listed in EN 1993-1-6 (D-1.4).

The shear buckling checks for tubular piles may be ignored if:

$$\frac{r}{t} \leq 0.16\left(\frac{E}{f_y}\right)^{0.67} \tag{9.26}$$

Otherwise, the design check shall satisfy the following conditions

$$\tau_{x\theta,Ed} \leq \tau_{x\theta,Rd} \tag{9.27}$$

$\tau_{x\theta,Ed}$ is design shear stress,

$\tau_{x\theta,Rd}$ is design resistance of buckling shear stress

$$\tau_{x\theta,Ed} = V/\pi.r.t \tag{9.28}$$

9.5.5 Computation of $\sigma_{x\theta,Rd}$ for tubular piles

i) Evaluation of critical elastic shear buckling stress $\tau_{x\theta,Rcr}$ using following expression;

$$\tau_{x\theta,Rcr} = 0.75\;E\;C_\tau\sqrt{\frac{1}{\omega}}(t/r) \tag{9.29}$$

Where,

t is the thickness of tube

r is the mean radius of cell

ii) Value of C_τ shall be obtained from Table- 9.12.

Table 9.12: Values of C_τ based on classification of cylinder (tubular pile) with respect to dimensionless length parameter;

Description of cylinder	**Boundary conditions**	C_τ
Long cylinder	$\omega > 8.7\,r/t$	$\frac{1}{3}\sqrt{\frac{\omega.t}{r}}$
Medium length cylinder	$10 \leq \omega \leq 8.7\,r/t$	1.0
Short cylinder	$\omega < 10$	$\sqrt{1+\frac{42}{\omega^2}}$

iii) Computation of relative shell slenderness parameter $\bar{\lambda}_\tau$ for shear buckling

$$\bar{\lambda}_\tau = \sqrt{\left(f_y/\sqrt{3}\right)/\tau_{x\theta,Rcr}} \tag{9.30}$$

iv) Computation of reduction factor χ_τ as a function of the relative slenderness of the shell $\bar{\lambda}_\tau$:

$$\chi_\tau = 1 \quad \text{when } \bar{\lambda}_\tau \leq \bar{\lambda}_0 \tag{9.31}$$

$$\chi_\tau = 1 - \beta\left(\frac{\bar{\lambda}_\tau - \bar{\lambda}_0}{\bar{\lambda}_p - \bar{\lambda}_0}\right)^\eta \quad \text{when } \bar{\lambda}_0 < \bar{\lambda}_\tau < \bar{\lambda}_p \qquad (9.32)$$

$$\chi_\tau = \frac{\alpha_\tau}{\bar{\lambda}_\tau} \quad \text{when } \bar{\lambda}_p \leq \bar{\lambda}_\tau \qquad (9.33)$$

Where: α_τ is the elastic imperfection reduction factor

β is the plastic range factor; $\beta = 0.60$;

η is the interaction exponent; $\eta = 1.0$

$\bar{\lambda}_{\tau 0}$ is the squash limit of relative slenderness; $\bar{\lambda}_{\tau 0} = 0.40$

$\bar{\lambda}_p$ is the plastic limit of relative slenderness to be determined from expression 9.34.

$$\bar{\lambda}_p = \sqrt{\frac{\alpha_\tau}{1 - \beta}} \qquad (9.34)$$

The shear elastic imperfection factor α_τ is provided in Table 9.13;

Table 9.13: Values of α_τ according to fabrication quality

Fabrication tolerance quality class	Description	α_τ
Class A	Excellent	0.75
Class B	High	0.65
Class C	Normal	0.50

v) Computation of axial Buckling resistance $\sigma_{x\theta,Rd}$ using following expression

$$\sigma_{x\theta,Rd} = \frac{\chi_\tau \cdot f_y/\sqrt{3}}{\gamma_{M1}} \qquad (9.35)$$

9.5.6 Verification of Buckling against Circumferential (hoop) compression

The circumferential pressure due to soil and water is not constant along the perimeter (and the length) of the tube and the maximum value of pressure shall be adopted for the verification of stress. The design rules given in EN 1993-1-6 do not take into account the stiffening effect of the soil located both inside (plug effect) and outside (embedded part of the retaining wall or pile) of the driven tube.

However, if tubular pile is filled with sand, the checking for circumferential stresses for buckling of pipe due to external hydrostatic pressures & soil pressure above embedment depth may be omitted. In case of empty or partially filled tube, the circumferential stress (σ_θ)in pile due to differential water pressure ($\Delta \mathrm{P_w}$)can be expressed as;

$$\sigma_\theta = \Delta P_w . r/t \tag{9.36}$$

The design check shall satisfy the following conditions

$$\sigma_{\theta,Ed} \leq \sigma_{\theta,Rd} \tag{9.37}$$

$\sigma_{\theta,Ed}$ is design circumferential stress,

$\sigma_{\theta,Rd}$ is design resistance of buckling against circumferential stress

9.5.7 Computation of $\sigma_{\theta,\mathrm{Rd}}$ for tubular piles

i) Evaluation of elastic critical circumferential buckling stress ($\sigma_{\theta,Rcr}$) using following expression;

Table 9.14: Values of $\sigma_{\theta,Rcr}$ based on classification of cylinder;

Description of cylinder	**Boundary conditions**	$\boldsymbol{\sigma_{\theta,Rcr}}$
Long cylinder	$\frac{\omega}{C_\theta} > 1.63\, r/t$	$E\left(\frac{t}{r}\right)^2\left[0.275 + 2.03\left(\frac{C_\theta . r}{\omega\, .\, t}\right)^4\right]$
Medium length cylinder	$20 \leq \frac{\omega}{C_\theta} \leq 1.63\, r/t$	$0.92\, E\left(\frac{C_\theta}{\omega}\right)(t/r)$
Short cylinder	$\frac{\omega}{C_\theta} < 20$	$0.92\, E\left(\frac{C_{\theta s}}{\omega}\right)(t/r)$

C_θ and $C_{\theta s}$ are the external pressure buckling factor for cylinder as given in Table 9.15 and Table 9.16.

Table 9.15: Values of C_θ (External pressure buckling factors for medium-length tubes)

Case	Boundary condition for pile ends fixidity		C_θ
1	end 1 end 2	BC 1 BC 1	1.5
2	end 1 end 2	BC 1 BC 2	1.25
3	end 1 end 2	BC 2 BC 2	1.0
4	end 1 end 2	BC 1 BC 3	0.6

Table 9.10: Values of $C_{\theta s}$(External pressure buckling factors for short tubes)

Case	Boundary condition for pile ends fixidity		$C_{\theta s}$
1	end 1 end 2	BC 1 BC 1	$1.5+\frac{10}{\omega^2}-\frac{5}{\omega^3}$
2	end 1 end 2	BC 1 BC 2	$1.25+\frac{8}{\omega^2}-\frac{4}{\omega^3}$
3	end 1 end 2	BC 2 BC 2	$1.0+\frac{3}{\omega^{1.35}}$
4	end 1 end 2	BC 1 BC 3	$0.6+\frac{1}{\omega^2}-\frac{0.3}{\omega^3}$

ii) Computation of relative shell slenderness parameter $\bar{\lambda}_\theta$ for axial stress

$$\bar{\lambda}_\theta = \sqrt{f_y/\sigma_{\theta,Rcr}} \qquad (9.38)$$

iii) Computation of reduction factor χ_θ due to circumferential buckling stress based on relative slenderness $\bar{\lambda}_\theta$:

$$\chi_\theta = 1 \qquad \text{when } \bar{\lambda}_\theta \leq \bar{\lambda}_0 \qquad (9.39)$$

$$\chi_\theta = 1 - \beta\left(\frac{\bar{\lambda}_\theta - \bar{\lambda}_0}{\bar{\lambda}_p - \bar{\lambda}_0}\right)^\eta \qquad \text{when } \bar{\lambda}_0 < \bar{\lambda}_\theta < \bar{\lambda}_p \qquad (9.40)$$

$$\chi_\theta = \frac{\alpha_\theta}{\bar{\lambda}_\theta} \qquad \text{when } \bar{\lambda}_p \leq \bar{\lambda}_\theta \qquad (9.41)$$

Where: α_θ is the elastic imperfection reduction factor

β is the plastic range factor ; $\beta = 0.60$;

η is the interaction exponent; $n = 1.0$

$\bar{\lambda}_0$ is the squash limit of relative slenderness; $\bar{\lambda}_0 = 0.40$

$\bar{\lambda}_p$ is the plastic limit of relative slenderness to be determined from expression 9.42.

$$\bar{\lambda}_p = \sqrt{\frac{\alpha_\theta}{1-\beta}} \qquad (9.42)$$

The shear elastic imperfection factor α_θ is provided in Table 9.17;

Table 9.17: Values of α_θ *based on* fabrication quality

Fabrication tolerance quality class	Description	α_θ
Class A	Excellent	0.75
Class B	High	0.65
Class C	Normal	0.50

iv) Computation of axial Buckling resistance using following expression

$$\sigma_{\theta,Rd} = \chi_\theta \cdot f_y/\gamma_{M1}; \qquad (9.43)$$

Depending on the loading and stressing situation, following checks for single membrane stress should be carried out as explained above:

$$\sigma_{x,Ed} \leq \sigma_{x,Rd}$$
$$\sigma_{\theta,Ed} \leq \sigma_{\theta,Rd}$$
$$\tau_{x\theta,Ed} \leq \tau_{x\theta,Rd}$$

Finally, Eurocode3 transpires an approach of superposing the maximum values of the different stress components to work out the resultant stress in tubular piles. If more than one of the three buckling-relevant membrane stress components is present under the actions under consideration, the following interaction check for the combined membrane stress state should be carried out:

$$\left(\frac{\sigma_{x,Ed}}{\sigma_{x,Rd}}\right)^{k_x} - K_i\left(\frac{\sigma_{x,Ed}}{\sigma_{x,Rd}}\right)\left(\frac{\sigma_{\theta,Ed}}{\sigma_{\theta,Rd}}\right) + \left(\frac{\sigma_{\theta,Ed}}{\sigma_{\theta,Rd}}\right)^{k_\theta} + \left(\frac{\tau_{x\theta,Ed}}{\tau_{x\theta,Rd}}\right)^{k_\tau} \leq 1 \qquad (9.44)$$

Where $\sigma_{x,Ed}$ or $\sigma_{\theta,Ed}$ are the compressive stresses. If it is tensile in nature, its value should be taken as zero in expression (9.33).

If tube is filled up with sand, the expression (9.33) can be modified as;

$$\left(\frac{\sigma_{x,Ed}}{\sigma_{x,Rd}}\right)^{k_x} + \left(\frac{\tau_{x\theta,Ed}}{\tau_{x\theta,Rd}}\right)^{k_\tau} \leq 1 \qquad (9.45)$$

The values of the buckling interaction parameters k_x , k_θ and k_τ are

$$K_x = 1.25 + 0.75\chi_x \qquad (9.46)$$
$$K_\theta = 1.25 + 0.75\chi_\theta \qquad (9.47)$$
$$K_\tau = 1.25 + 0.55\chi_\tau \qquad (9.48)$$
$$K_i = \left(\chi_x . \chi_\theta\right)^2 \qquad (9.49)$$

Example: 9.2

A thin cell tubular steel pile (class-4 section) having effective length=25m; outer diameter = 1200mm; cell thickness = 16mm; steel grade - 420GP, is installed under a fixed type platform. For ultimate limit state design of tubular pile, the applicable normal load, moment and shear forces are 6150KN, 3150KN.m and 280KN respectively. Verify the load carrying *capacity of pile, considering the overall buckling effect in accordance with Eurocode3.*

Solution:

Checking of Stresses in Tubular Pile due to Overall Buckling Effect

Ultimate Loads on Tubular Pile

Normal Axial Force [N]	N	=	6150.0	KN
Normal Shear Force [V]	V		280.0	KN
Bending Moment [Mz]	Mz		3150.0	KN.m

Properties of Tubular Pile

Eff. Length of Pile	*l*	=	25.0	M
Dia. Of Pile		=	1200.0	mm
Thickness of Shell	T	=	16.0	mm
Corrosion allowance (50 Years)		=	2.75	mm

After corrosion loss in pile section

Net OD of tubular pile	Do	=	1197.3	mm
Net inside dia. of tubular Pile	Di	=	1184.0	mm
Mean Dia.	D	=	1190.6	mm
Mean Radius	r	=	595.3	mm
Net Shell thickness	t	=	13.3	mm
Yield stress	fy	=	420.0	N/mm^2
Dimensionless length parameter $\omega = l/\sqrt{r.t}$	ω	=	281.5	
Area of C/S of pile	Ax	=	24780.5	mm^2
Iyy = Izz		=	4.39E+09	mm^4
Sy = Izz / y		=	5532118.9	mm^3
Axial Stress	σ_{x1}	=	5.65	N/mm^2
Bending Stress	σ_{x2}	=	213.53	N/mm^2

$\sigma_{X,Ed} = \sigma_{x1} + \sigma_{x2}$ $\sigma_{x,Ed}$ = 219.18 N/mm^2

Shear Stress $\tau_{x\theta,Ed} = (V / \pi.r.t)$ $\tau_{x\theta,Ed}$ = 11.30 N/mm^2

A Verification of Buckling against Axial Compression

Modulus of Elasticity	E	=	210000	N/mm^2
Yield Stress of Steel (Tubular Pile)	fy		420	N/mm^2
r/t		=	44.93	
0.03 E/fy		=	15.00	
r/t > 0.03 E/fy - Axial Buckling check is required				
$\omega = l/\sqrt{r.t}$	ω	=	281.49	
ω > 0.5*r/t			Long Cylinder	
Fixidity Condition of Pile Top End BC-1 (Hinged) & Bottom BC-2 (fixed)]	C_{xb}	=	3.0	
$1 + \frac{0.2}{C_{xb}}\left[1 - 2\,\omega.{}^{t}/_{r}\right] \ngtr 0.6$	C_x	=	0.231	-
0.6 . E . Cx . t/r	$\sigma_{x,Rcr}$	=	648.7	N/mm^2
$\bar{\lambda}_x = \sqrt{f_y/\sigma_{x,Rcr}}$	λ_x	=	0.805	
Manufacturing Tolerance Quality Parameter	Q	=	25	
$\Delta w_k = \frac{1}{Q}\sqrt{\frac{r}{t}}.t$	Δw_k	=	3.553	
$\alpha_x = \frac{0.62}{1+1.91(\Delta w_k/t)^{1.44}}$	α_x	=	0.482	
	λo	=	0.20	
	β	=	0.60	
	η	=	1.0	
$\bar{\lambda}_p = \sqrt{\frac{\alpha_x}{1-\beta}}$	$\bar{\lambda}_p$	=	1.097	

	$\bar{\lambda}_0 < \bar{\lambda}_x < \bar{\lambda}_p$				
	$\chi_x = 1 - \beta \left(\frac{\bar{\lambda} - \bar{\lambda}_0}{\bar{\lambda}_p - \bar{\lambda}_0} \right)^{\eta}$	χ_x	=	0.596	
		γ_{M1}	=	1.10	
	$\sigma_{x,Rd} = \chi_x \cdot f_y / \gamma_{M1}$	$\boldsymbol{\sigma_{x,Rd}}$	=	**227.47**	N/mm^2
	$\sigma_{x,Ed} < \sigma_{x,Rd}$			**(O.K)**	
B	**Verification of Buckling against Shear Forces**				
	V / π.r.t	$\tau_{x\theta,Ed}$	=	11.30	N/mm^2
	$0.16 \left(\frac{E}{f_y} \right)^{0.67}$		=	10.29	
	$\frac{r}{t} < 0.16 \left(\frac{E}{f_y} \right)^{0.67}$ Buckling check against shear force is required				
	8.7 * r/t 10 < ω < 8.7 r/t => Med. Length cylinder		=	390.88	
		C_τ	=	1.0	
	$\tau_{x\theta,Rcr} = 0.75\, E\, C_\tau \sqrt{\frac{1}{\omega}}\, (t/r)$	$\tau_{x\theta,Rcr}$	=	208.94	N/mm^2
	$\bar{\lambda}_\tau = \sqrt{\left(f_y/\sqrt{3}\right)/\tau_{x\theta,Rcr}}$	$\bar{\lambda}_\tau$	=	1.08	
	Shear elastic imperfection factor (Class-B)	α_τ	=	0.65	
		$\bar{\lambda}_{\tau 0}$	=	0.40	
		β	=	0.60	
		η	=	1.0	
	Plastic limit of relative slenderness $\bar{\lambda}_p = \sqrt{\frac{\alpha_\tau}{1 - \beta}}$	$\bar{\lambda}_p$	=	1.275	

$\bar{\lambda}_0 < \bar{\lambda}_x < \bar{\lambda}_p$				
$\chi_\tau = 1 - \beta \left(\frac{\bar{\lambda}_\tau - \bar{\lambda}_0}{\bar{\lambda}_p - \bar{\lambda}_0} \right)^\eta$	χ_τ	=	0.535	
$\sigma_{x\theta,Rd} = \frac{\chi_\tau \cdot f_y/\sqrt{3}}{\gamma_{M1}}$	$\sigma_{x\theta,Rd}$	=	**118.03**	**N/mm²**
$\tau_{x\theta,Ed} < \tau_{x\theta,Rd}$			**(O.K)**	

C Verification of Buckling against Circumferential Compression

Differential water head in tubular piles	ΔH_w	=	2.50	M
Differential water Pressure	ΔP_w	=	0.025	N/mm²
$\sigma_{\theta,Ed} = \Delta P_W \cdot r/t$	$\boldsymbol{\sigma_{\theta,Ed}}$	=	**1.13**	N/mm²
	C_θ	=	1.00	
1.63*r/t		=	73.23	
	$\frac{\omega}{C_\theta}$	=	281.49	
$\frac{\omega}{C_\theta} > 1.63\, r/t;$ Hence, Long Cylinder				
$E\left(\frac{t}{r}\right)^2 \left[0.275 + 2.03\left(\frac{C_\theta . r}{\omega\ .\ t}\right)^4\right]$	$\sigma_{\theta,Rcr}$	=	28.75	N/mm²
$\bar{\lambda}_\theta = \sqrt{f_y/\sigma_{\theta,Rcr}}$	$\bar{\lambda}_\theta$	=	3.82	
For Class-B section	α_θ $\bar{\lambda}_0$ β η	= = = =	0.65 0.40 0.60 1.00	
$\bar{\lambda}_p = \sqrt{\frac{\alpha_\theta}{1 - \beta}}$	$\bar{\lambda}_p$	=	1.27	
$\chi_\theta = \frac{\alpha_\theta}{\bar{\lambda}_\theta}$	χ_θ	=	0.51	

$\sigma_{\theta,Rd} = \chi_{\theta} \cdot f_y/\gamma_{M1}$	$\sigma_{\theta,Rd}$	=	194.69	N/mm^2
$\sigma_{\theta,Ed} \leq \sigma_{\theta,Rd}$			**(O.K)**	
$K_x = 1.25 + 0.75\chi_x$	K_x	=	1.697	
$K_\theta = 1.25 + 0.75\chi_\theta$	K_θ	=	1.632	
$K_\tau = 1.25 + 0.55\chi_\tau$	K_τ	=	1.884	
$K_i = \left(\chi_x \cdot \chi_\theta\right)^2$	K_i	=	0.300	
$\left(\frac{\sigma_{x,Ed}}{\sigma_{x,Rd}}\right)^{k_x}$		=	0.940	
$K_i\left(\frac{\sigma_{x,Ed}}{\sigma_{x,Rd}}\right)\left(\frac{\sigma_{\theta,Ed}}{\sigma_{\theta,Rd}}\right)$		=	-1.71E-03	
$\left(\frac{\sigma_{\theta,Ed}}{\sigma_{\theta,Rd}}\right)^{k_\theta}$		=	2.25E-04	
$\left(\frac{\tau_{x\theta,Ed}}{\tau_{x\theta,Rd}}\right)^{k_\tau}$		=	1.20E-02	
$\left(\frac{\sigma_{x,Ed}}{\sigma_{x,Rd}}\right)^{k_x} - K_i\left(\frac{\sigma_{x,Ed}}{\sigma_{x,Rd}}\right)\left(\frac{\sigma_{\theta,Ed}}{\sigma_{\theta,Rd}}\right) + \left(\frac{\sigma_{\theta,Ed}}{\sigma_{\theta,Rd}}\right)^{k_\theta} + \left(\frac{\tau_{x\theta,Ed}}{\tau_{x\theta,Rd}}\right)^{k_1}$		=	**0.95**	**< 1**
				(O.K.)

9.6 Design of Combi-Walls

Combined walls or combi-walls are comprise steel bearing piles, acting as structural supports in conjunction with secondary sheet piles acting as intermediate earth retaining infill elements. These walls provide increased stiffness or if necessary, provide improved bearing capacity. The combined wall can be formed from alternating tubular and sheet piles. The tubular piles take vertical loads and provide stiffen to the wall. For HZ walls system, proprietary I or H sections are interlocked with Z profile sheets using a specifically designed hot-rolled connector.

9.6.1 Standard Conditions for Design of Combi-Walls

The standard approach (Eurocode EN 1993-5) for assessment of loads in combi-walls assumes that the primary elements bear the earth pressures and the secondary elements bear the water pressure. This approach is based on the assumption that arch formed within the soil, transfers the loads to the primary elements. However, for sheet piles used as secondary elements, further verification may be omitted if all the following conditions are met:

a) wall thickness of the secondary sheet piles ≥ 10mm;
b) pressure difference acting on the sheet piles: ≤ *40kN/m^2,* corresponding to 4 m differential water head;
c) maximum clearance between the primary elements is 1.8m for U-piles and 1.5m for Z-piles.

If maximum clearance between primary or king piles, exceeds the above limits, arching effect of soil between rigid piles shall be verified and secondary sheet pile section shall be verified accordingly. Arching mechanism in soil is also a function of shear strength of soil which may be verified for actual sub soil condition as given below.

9.6.2 Verification for Arching of Soil

Soil Arching is defined as the transfer of pressure from a yielding mass of soil onto adjacent stationary parts of soil. In the transition zone between moving and stationary soil masses, shear stresses develop due to the relative displacement between the two masses. Since the shearing resistance tends to maintain the yielding mass in its original position, the pressure on the support of the yielding part is reduced while the pressure on the adjoining stationary part in increased. It occurs whenever there is a localized displacement along any restraining structure, horizontal or vertical, which confines a soil mass.

In soil arching, passive arching occurs when the soil is more compressible than the structure and as a result the soil undergoes large displacements. In this latter case, assuming the structural deformations are uniform, the stresses are highest at the edges and lowest at the centreline of the moving soil mass. When analysing soldier pile walls (or combi wall with secondary elements), passive arching can be developed, as the soil masses are restrained by stiffer structural elements (king piles or supports)

The development of the Wang-Reese equations for evaluating spacing of king piles to develop the passive resistance in cohesionless soils and cohesive soils is presented here. This is largely based on information and details provided in FHWA-RD-97-130 (1998).

9.6.3 Arching Effect in Cohesionless Soils

For case of soil arching, the Wang-Reese equations for ultimate passive resistance of cohesionless soils develop a potential failure mechanism considering an overlapping wedge failure for deep or closely-spaced shafts as shown in **Figure 9.5**.

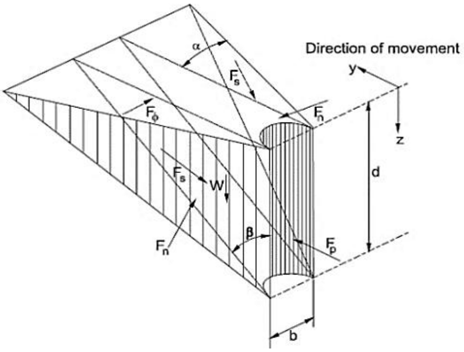


Figure 9.5 Intersecting failure wedges in sand

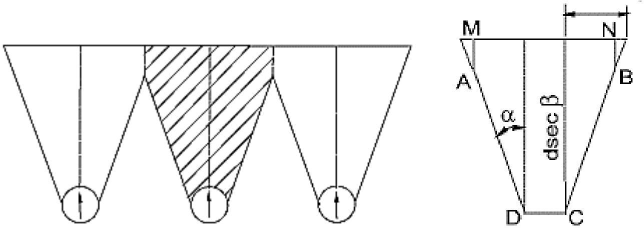


Figure 9.6 Intersecting failure wedges for soldier beams in sand (after Wang and Reese, 1986).

Figure 9.6 shows individual failure wedges intersecting. This may occur, based on the analysis method, for a case where adjacent soldier beams are relatively close to each other or where the depth of the bottom of the soldier beam is relatively large. Equation 9.50 gives the depth of the intersection, di, of adjacent wedges.

$$di = d - Sc / 2.\tan\alpha.\tan\beta \qquad (9.50)$$

Where:

Sc is the clear spacing between adjacent soldier beams

$\alpha = \phi'$ for dense sands, $\phi'/3$ to $\phi'/2$ for loose sands.

$\beta = 45+\phi'/2$; and

ϕ'= drained friction angle of the soil;

d = depth of the bottom of the soldier beam (Ref Figure 9.5);

When di is positive, the failure wedges of retained soil intersect. If di is negative, the failure wedges do not intersect and arching action is not effective.

9.6.4 Arching Effect in Cohesive Soils

Equation 9.51 gives the critical spacing, Sc for cohesive soil, where the behaviour changes from single beam behaviour to group behaviour.

$$S_c = \frac{2.83 * S_u * d}{\gamma d + 6S_u} \qquad (9.51)$$

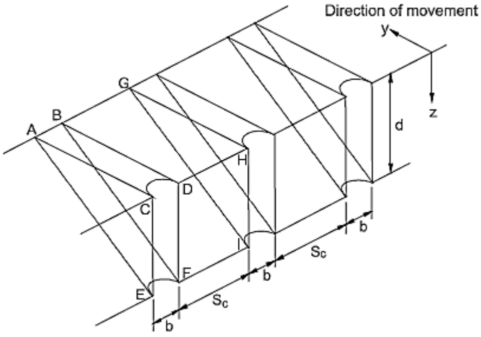


Figure 9.7 Intersecting failure wedges for soldier beams in clay

9.6.5 Verification for Pressures on Secondary Sheet Piles

If wall thickness of the secondary sheet piles <10mm or pressure difference acting on the sheet piles is more than $40kN/m^2$, the sectional properties of secondary sheet pile should be verified for actual induced moments as given in section 9.8.1.

9.7 Combined Sheet Piles: H + Z

H-Z combined wall comprise of steel king piles acting as structural support in conjunction with secondary sheet piles acting as diaphragms between king piles. This type of wall most often employed when regular sheet piles (consisting of Z or U-profile) are not strong enough to support the required load. In place of king piles, H-piles can be used and double-Z pile are generally used as infill or secondary sheet piles. To form H-Z wall system, proprietary H-sections, interlocked with Z profile piles and specifically designed hot-rolled connectors are used. See Fig 9.8.

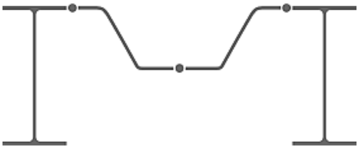


Figure 9.8 H+Z combi-wall

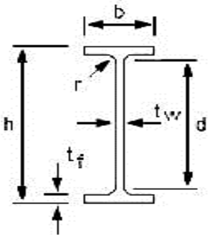
Generally, I-sections used as primary elements in combined wall are class 3 sections according to EN 1993-1-1, and sections may be verified according to the procedure outlined below;

9.7.1 Design Verification Method

According to Eurocode EN 1993-5 (1), the following simplified procedure may be followed, if clear spacing between king piles and lateral pressure on secondary sheet piles are within limits as specified in section 9.6.1. The adequacy of H-sections shall be verified by applying the following check;

Interaction between overall bending, axial forces and local plate bending in the flanges due to lateral loads.

i) Cross-section Classification (flanged Section)

	Class 3	Limiting Values	
Flange Outstand	$c/t_f \leq 14\ \varepsilon$		
Web in bending	$d/t_f \leq 142\varepsilon$		
Web in compression	$d/t_f \leq 42\ \varepsilon$		

Where, c = (b - t_w - 2r)/2 and d = h - 2t_f - 2r

ii) Shear buckling resistance of web

The shear buckling resistance of the web may be verified in accordance with *EN1993-1-1 §6.2.6(6)*. Shear buckling resistance without additional measures is proved when:

$$h \,/\, t_w \leq 72 \cdot \varepsilon \,/\, \eta \tag{9.52}$$

The shear area factor, η is taken as:

η = 1.20 for $f_y \leq 460\ N/mm^2$, and

η = 1.0 othewise

iii) Compression (EN1993-1-1 §6.2.4)

The critical cross-section shall be verified for compressive axial force N_{Ed} in accordance with *EN1993-1-1 §6.2.4*:

$$\frac{N_{Ed}}{N_{cRd}} \leq 1.0 \tag{9.53}$$

Where, N_{Ed} is the design compressive axial force.

For the case of class 1, 2, or 3 cross sections, the compression resistance N_{cRd} *should be* estimated as:

$$N_{c\,Rd} = \frac{A.f_y}{\gamma_{M0}} \tag{9.54}$$

iv) Bending moment (EN1993-1-1 §6.2.5)

The design check for bending moment shall generally satisfy the following conditions;

$$\frac{M_{Ed}}{M_{c,Rd}} \leq 1.0 \qquad (9.55)$$

$M_{c,Rd} = W_{el} f_y / \gamma_{M0}$ (for class 3 sheet pile sections)

The elastic section modulus values, *W*$_{el,}$ are part of the combi-wall definition.

For class 3 cross-sections the design resistance $\mathrm{M_{c\,Rd}}$ for bending about one principal axis is estimated as the corresponding elastic bending resistance $\mathrm{M_{el\,Rd}}$ on the basis of the elastic section modulus $\mathrm{W_{el,min.}}$ corresponding to the fibre with the maximum elastic stress.

v) Shear Check

For class 3 cross-sections, the design elastic shear resistance should be verified in accordance with EN1993-1-1 §6.2.6(4). The maximum shear stress $\mathrm{\tau_{Ed}}$ for the critical point of the cross-section is verified on the basis of the Von Mises yield criterion:

$$\frac{\tau_{Ed}}{f_y / [\sqrt{3} \gamma_{M0}]} \leq 1.0 \qquad (9.56)$$

The shear stress $\mathrm{\tau_{Ed}}$ should be evaluated on the basis of elastic theory as specified in EN1993-1-1 §6.2.6:

$$\tau_{Ed} = V_{Ed} . S / (I_y \ . \ t_f) \qquad (9.57)$$

Where, *S* is the first moment of area about the centroidal axis of the part of the cross-section between the examined point and the boundary of the cross-section, I_y is the corresponding second moment of area of the whole cross-section, and t_f is the thickness of the flange.

vi) Bending and shear (EN1993-1-1 §6.2.8)

The effect of the shear force on the moment resistance should be examined in accordance with *EN1993-1-1 §6.2.8*. In the absence of transverse stress (i.e. $\sigma_{z\,Ed}$ = 0) the yield criterion may be expressed as:

$$\sigma_v = [\, \sigma_{x\,Ed}{}^2 + 3. \tau_{Ed}{}^2]^{1/2} \leq \ f_y / \gamma_{Mo} \qquad (9.58)$$

Where, σ_v is the equivalent Von Mises stress, $\sigma_{x\,Ed}$ is the longitudinal stress, and τ_{Ed} is the shear stress. The longitudinal stress $\sigma_{x\,Ed}$ is evaluated on the basis of elastic theory:

$$\sigma_{x\,Ed} = \frac{N_{Ed}}{A} + \frac{M_{y\,Ed}.Z.}{I_y} + \frac{M_{z\,Ed}.\ Y.}{I_z} \qquad (9.59)$$

where N_{Ed} is design axial force,$M_{y\,Ed}$, $M_{z\,Ed}$ is the design bending moment about the major axis Y-Y and the minor axis Z-Z respectively, I_y and I_z is the second moment of area of the cross-section for bending about the major axis Y-Y and the minor axis Z-Z respectively, and Y, Z are the horizontal and vertical distance of the examined point from the centroid of the cross-section respectively.

vii) Bending and axial force (EN1993-1-1 §6.2.9)

For class 3 cross-sections the longitudinal stress σ_x in combined bending moment and axial force must satisfy the yield criterion as specified in *EN1993-1-1 §6.2.9.2* as

$$\sigma_{x\,Ed} \leq f_y/\gamma_{Mo}$$

The longitudinal stress $\sigma_{x\,Ed}$ is evaluated on the basis of elastic theory:

$$\sigma_{x\,Ed} = \frac{N_{Ed}}{A} \pm \frac{M_{y\,Ed}.Z.}{I_y} \pm \frac{M_{z\,Ed}.\ Y.}{I_z} \qquad (9.60)$$

viii) Flexural buckling (EN1993-1-1 §6.3.1.3)

The compression member is verified against flexural buckling in accordance with *EN1993-1-1 §6.3.1* as follows:

$$\frac{N_{Ed}}{N_{b\,Rd}} \leq 1.0 \qquad (9.61)$$

Where, $N_{b\,Rd}$ is the design buckling resistance of the compression member given in *EN1993-1-1 §6.3.1.1(3)* for class 1, 2 and 3 cross-sections:

$$N_{b\,Rd} = \frac{\chi \cdot A \cdot f_y}{\gamma_{M1}} \qquad (9.62)$$

The reduction factor χ due to flexural buckling is calculated for the major and the minor bending axes. The reduction factor, χ for the relevant buckling mode is taken as:

$$\chi = \frac{1}{\emptyset + \sqrt{\emptyset^2 - \bar{\lambda}^2}} \leq 1.0 \qquad (9.63)$$

Where, the factor, *Φ* and the non-dimensional slenderness, $\bar{\lambda}$ are taken as:

$$\emptyset = 0.5\left[1 + \alpha(\bar{\lambda} - 0.2) + \bar{\lambda}^2\right] \qquad (9.64)$$

$$\bar{\lambda} = \frac{L_{cr}}{i}\frac{1}{\lambda_1} \text{ for Class 1, 2 and 3 cross-sections} \qquad (9.65)$$

Where, $\lambda_1 = \pi\sqrt{\frac{E}{f_y}} = 93.3\,\varepsilon$ (9.66)

L_{cr} is the buckling length in the buckling plane

i is the radius of gyration about the relevant axis

α is an imperfection factor as given in Table below.

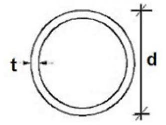
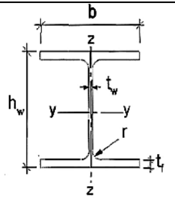
Buckling curve	a_0	a	B
Imperfection factor α	0.13	0.21	0.34

Table 9.18 (Ref. EN1993-1-1 §6.2.9 - Table 6.2)

$\bar{\lambda}$		**0.0**	**0.2**	**0.4**	**0.6**	**0.8**	**1.0**	**1.2**	**1.4**	**1.6**	**1.8**	**2.0**	**2.2**	**2.4**	**2.6**	**2.8**	**3.0**
χ	**a**	1.0	1.0	0.97	0.93	0.87	0.73	0.58	0.44	0.36	0.28	0.24	0.20	0.16	0.14	0.12	0.11
	b	1.0	1.0	0.96	0.92	0.80	0.66	0.54	0.42	0.34	0.27	0.23	0.19	0.16	0.14	0.11	0.10
	a_0	1.0	1.0	0.95	0.89	0.72	0.60	0.48	0.38	0.31	0.26	0.21	0.18	0.15	0.13	0.11	0.10

H-piles having h/b ≤ 1.2 & $t_f \leq 40\ mm$; values of reduction factor, χ for different grades of steel can be adopted from table 9.19.

Table 9.19 Reduction factor, χ for pile sections

Cross section		Limits	Buckling about axis	Buckling curve for steel Grade	
				S-235, S-275 S-355, S-420	S-460
Tubular Pile		Hot rolled	any	A	a_0
		Cold formed	any	C	c
Rolled Section		h/b ≤ 1.2 & $t_f \le 40\ mm$	y - y	A	a_0
			z - z	B	a_0

ix) Axial Buckling Check

The axial buckling check for each section shall satisfy:

$$N_{Ed}/N_{b,Rd} \leq 1.0 \tag{9.67}$$

Where, the design compression resistance, $N_{b,Rd}$ for Class 1, 2, 3, sections are taken as:

$$N_{b,Rd} = \frac{\chi A\, f_y}{\gamma_{M1}} \tag{9.68}$$

The reduction factor, χ for the relevant buckling mode shall be adopted from equation 9.32:

$$\chi = \frac{1}{\emptyset + \sqrt{\emptyset^2 - \lambda^{-2}}} \leq 1.0 \tag{9.69}$$

The axial buckling check is not ignored if the value of $\bar{\lambda}$ and $\frac{N_{Ed}}{N_{cr}}$ is:

$$\bar{\lambda} \leq 0.2\ or\ \frac{N_{Ed}}{N_{cr}} \leq 0.04 \tag{9.70}$$

Where, $N_{cr} = EI.\beta_D.\mu^2/l^2$

9.7.2 Verification of Local Plate Bending of the Flanges

Local plate bending in the flanges should be verified for a cross-section through the flange located at the beginning of the fillet taking into account the design forces introduced via the connectors, see Fig 9.9.

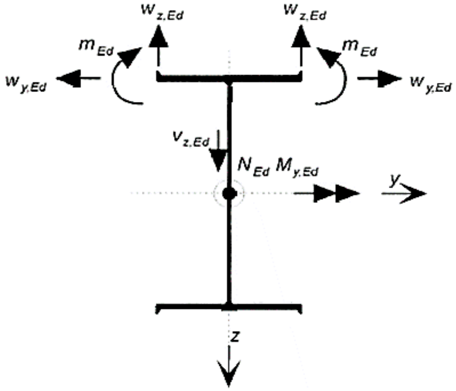


Figure 9.8 Force Diagram for H-Pile

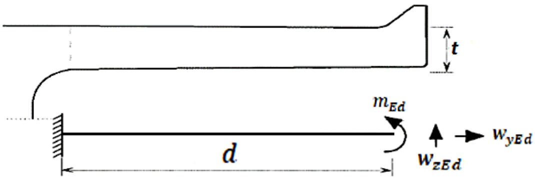


Figure 9.9 Force Diagram in flange of H-Pile

Using the force diagram in Fig-9.11;

$$\frac{M_{Ed}}{M_{Rd}} + \left(\frac{N_{Ed}}{N_{Rd}}\right)^2 \leq 1 \tag{9.71}$$

Where, M_{Ed} and N_{ED} are the design action effects for plate bending given by

$$M_{Ed} = m_{Ed} + w_{zEd}d \quad and \quad N_{Ed} = w_{yEd} \tag{9.72}$$

M_{Rd} and N_{Rd} are the design values of the resistances for plate bending, given by:

$$M_{Rd} = 0.2875\, t^2 \frac{f_y}{\gamma_{M0}} \tag{9.73}$$

and $$N_{Rd} = t.f_y/\gamma_{M0} \tag{9.74}$$

Where, *t* is the flange thickness at the beginning of the fillet.

NOTES:

1) M_{Ed} , N_{Ed}, M_{Rd} *and* N_{Rd} *are to be taken per unit length.*
2) *The shear force interaction may be neglected.*
3) *Reference should be made to EN 1993-1-5 for the shear buckling verification of the webs.*
4) *Reference should be made to section 6.3.3 of EN1993-1-1 for the overall buckling verification.*

Up to a water pressure (or equivalent earth pressure in very soft soils) of *h* 10m head the interaction between overall action effects and local plate bending may be taken into account as follows:

The cross-sectional verification of the primary elements should be carried out according to 6.2.9.2 and 6.2.10 of EN 1993-1-1, taking into account reduced yield strength:

for h = 10m:	$f_{y,red} = 0.9\, f_y$
for *h* = 4 m: $f_{y,red}$	$f_{y,red} = 1.0\, f_y$
for 4 m < *h* < 10m:	linear interpolation

Example: 9.3

A high modulus combined wall section consist of H-Z piles as shown in figure E:9-1 installed for an anchored retaining wall in marine environments. Effective lengths of piles are 24m and a section property of pile is given in Table E: 9-3 below. For ultimate limit state design, the applicable normal load, moment and shear forces in piles are 482KN, 5950KN.m and 1900KN respectively. Verify the structural adequacy of pile for applied loads in accordance with Eurocode3.

Solution:

Design Parameters

Design compressive force (N_{ED}) = **482.0** KN

Design bending moment (M_{Ed}) = **5950.00** KN.m

Design shear force (V_{Ed}) = **1900.0** KN

Pile yield strength (f_y) = **420** N/mm^2

Pile Young's Modulus (Es) = **210000** N/mm^2

Partial material factor (γ_{M0}) = **1.00**

Design life = **50.00** years

For designing of wall, combination of sheet piles (combination of H-beams and Z-piles are considered.) The H-beam will act as a king pile to bear the contributory lateral and vertical loads and Z- piles will act as a diaphragm to transfer the water pressure on H-piles.

Corrosion allowances are provided in piles for loss of section in submerged zone. The summary of design loads and design section properties of H-piles are given below.

Design of H-pile Section

i) Section properties of pile

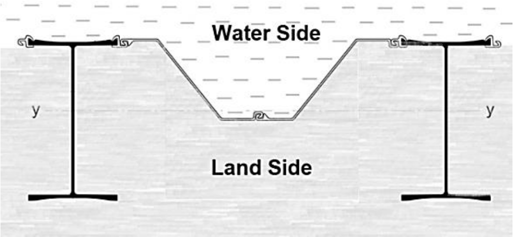
Sheet piling corrosion section loss :

Active face of sheet piling (splash zone) (protected by concrete encasing) = 0.00 mm

Active face of sheet piling (submerged zone) = 1.75 mm

Passive face of sheet piling = 0.600 mm

Pile Section = **HZ- 1080D**



Pile section properties :			**Full properties**		**Reduced properties** (after allowing for corrosion)	
Pile type		=	**H**		H	mm
Pile System width	B	=	**1.927**	mm	**1.927**	mm
Width of flange	b	=	**457**	mm	**457**	mm
Flange thickness	t_f	=	**30.7**	mm	**28.35**	mm (Sea side)
					29.50	mm (Land side)
Web thickness	t_w	=	**19**	mm	**17.80**	mm
Overall height of section	h	=	**1075**	mm	**1073**	mm
Moment of Inertia	Iy	=	**9E+05**	cm^4	**866066.33**	cm^4
Moment of Inertia	Iz	=	**46940**	cm^4	**44104.806**	
Plastic section modulus	W_{pl}	=	**-**	cm^3	**-**	
Elastic section modulus	W_{el}	=	**17340**	cm^3	**16229**	cm^3
Cross section	A	=	**473.0**	cm^2	**443.9**	cm^2
	EI	=	**2E+06**	$KN.m^2$	**1818739.3**	$KN.m^2$

ii) Shear Buckling of Web in H- pile

To satisfy the criteria unstiffned web buckling $\boldsymbol{h/t_w \leq 72.\varepsilon/\eta}$

h = **1073** mm

t_w = **17.80** mm

$\varepsilon = \sqrt{235/f_y}$ $\quad \varepsilon$ = **0.75**

η = **1.2**

h / t_w = **60.26**

$72\varepsilon / \eta$ = **44.88**

for, $h / t_w > 72\, \varepsilon / \eta$ **web shear buckling shall be varified**

iii) Buckling of flange in H- pile

To satisfy the criteria unstiffned flange buckling $\boldsymbol{b/t_f} \leq \mathbf{14}\, \varepsilon$

b = **457** mm

t_w = **17.80** mm

Flange outstand $c = (b - t_w)/2$ = **219.6** mm

(neglecting the fillet radius)

t_f = **28.35** mm

c/t_f = **7.75**

14ε = **10.47**

for, c/t_f < 14ε **Flange Outstand is satisfactory**

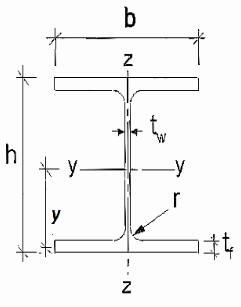
iv) Shear Capacity of section

For class-3 sections, maximum shear stress in beam should satisfy

Shear stress $\tau_{Ed} = V_{Ed}.S / I_y.t_f$; $\dfrac{\tau_{Ed}}{f_y/[\sqrt{3}\gamma_{M0}]} \leq 1.0$

Design shear force in wall = **1900.0** KN

Design shear force in H- pile (V_{Ed}) = **1900.0** KN



b = **457** mm

t_f = **28.35** mm

$y = h/2$ = **536.3** mm

$A = b_f . t_f$ = **12956** mm^2

$S = A.y$ = **6948600** mm^3

I_y = **866066** cm^4

$\tau_{Ed} = V_{Ed}.S / I_y.t_f$; = **53.77** N/mm^2

$f_y/\sqrt{3}\ \gamma_{M0}$ = **242.49** N/mm^2

$\tau_{Ed}\ /\ f_y/\sqrt{3}\ \gamma_{M0}$ = **0.22** < 1.0 **O.K**

v) Moment Resistance Capacity

Section modulus of H- Pile (W) = **16228.9** cm^3

Applied moment in wall = **5950.00** KN.m

Design bending moment in H-pile(M_{Ed}) = **5950.00** KN.m

$M_{cRd} = W_{el} f_y / \gamma_{M0}$ = **6816.14** KN.m

M_{Ed} / M_{cRd} = **0.873** **Moment capacity satisfactory**

vi) Check for Axial Compression

Applied compressive force in wall = **482.0** KN

Design compressive force in Pile (N_{ED}) = **482.0** KN

Area of cross-section of H-pile (A) = **443.9** cm^2

$N_{cRd} = A\, f_y / \gamma_{M0}$ = **21394.8** KN

$N_{Ed}/N_{cRd} < 1.0$ N_{Ed}/N_{cRd} = **0.0225** **Axial capacity satisfactory**

vii) Check for axial Buckling

Wall type = **Propped**

Unsupported length (L) = **24.0** m

Effective buckling length factor = **1.0**

Effective buckling length (l) = **24.0** m

$N_{cr} = E\, I\, \beta_D\, \pi^2 / l^2$ N_{cr} = **31164** KN

$N_{Ed}/N_{cr} < 0.04$ N_{ED}/N_{cr} = **0.0155**

$\lambda_1 = 93.3\, \varepsilon$ = **69.79**

radius of gyration (*i*)of H-pile

about y-y axis $i = \sqrt{I_y / A}$ or i = **44.23** cm

L_{cr} = **24.00** m

$\bar{\lambda} = \frac{L_{cr}}{i} \frac{1}{\lambda_1}$ $\bar{\lambda}$ = **0.78**

$\bar{\lambda}$ > **0.20** **Buckling resistance needs to be verified.**

reduction factor, χ = **0.956**

buckling resistance in compression

$N_{b,Rd} = \frac{\chi A f_y}{\gamma_{M1}}$ = **18992** KN

$N_{Ed}/N_{b,Rd}$ = **0.025** < 1.0

Section is satisfactory in axial buckling

viii) Combined Bending and Compression

Longitudinal stress $\sigma_{x,Ed}$ in combined bending moment and axial force must satisfy the yield criterion $\sigma_{xEd} \leq f_y/\gamma_{M0}$

f_y/γ_{M0} = **420.00** N/mm^2

Based on elastic theory,

$\sigma_{xEd} = {}^{N_{Ed}}/_{A} + M_{zEd}.z/I_y$

N_{Ed} = **482.00** KN

A = **443.88** cm^2

N_{Ed}/A = **10.86** N/mm^2

M_{zEd} = **5950.00** KN .m

I_y = **866066** cm^4

z = h/2 = **536.33** mm

$M_{zEd}.z/I_y$ = **368.46** N/mm^2

σ_{xEd} = **379.32** N/mm^2

$\sigma_{xEd}/f_y/\gamma_{M0}$ = **0.903** **<** **1.0 (O.K)**

ix) Combined Stress due to Bending and Shear

Design resistance in combined bending moment and shear force

$\sigma_v = \left[\sigma_{xEd}{}^2 + 3.\tau_{Ed}{}^2\right]^{.2} \leq {}^{f_y}/_{\gamma_{M0}}$

σ_{xEd} = **379.32** N/mm^2

τ_{Ed} = **53.77** N /mm^2

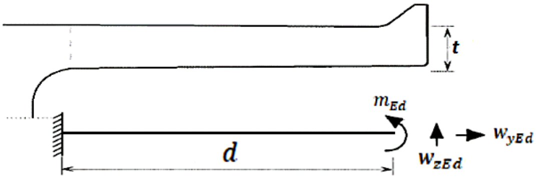
σ_v = **390.59** N /mm^2

f_y/γ_{M0} = **420.00** N /mm^2

$\sigma_v/f_y/\gamma_{M0}$ = **0.93** **<** **1.0 (O.K)**

x) Verification of Local Plate Bending of the Flanges

Local plate bending in the flanges should be verified as



$$\frac{M_{Ed}}{M_{Rd}} + \left(\frac{N_{Ed}}{N_{Rd}}\right)^2 \leq 1$$

$$M_{Ed} = m_{Ed} + w_{zEd} d \quad and \quad N_{Ed} = w_{yEd}$$

$$M_{Rd} = 0.2875\, t^2 \frac{f_y}{\gamma_{M0}} \text{ and } N_{Rd} = t.f_y/\gamma_{M0}$$

Outstand of flange	d	=	**219.0**	mm
Thickness of flange	t	=	**28.4**	mm
	m_{Ed}	=	**46.0**	KN.m
	W_{yEd}	=	**20.0**	KN
	W_{zEd}	=	**50.0**	KN
	M_{Ed}	=	**57.0**	KN.m
	$N_{Ed} = W_{yEd}$	=	**20.0**	KN
	$M_{Rd} = 0.2875\, t^2 \frac{f_y}{\gamma_{M0}}$	=	**94.0**	KN.m
	$N_{Rd} = t \cdot \frac{f_y}{\gamma_{M0}}$	=	**11907.0**	KN
	M_{Ed}/M_{Rd}	=	**0.606**	
	N_{Ed}/N_{Rd}	=	**0.0017**	
	$M_{Ed}/M_{Rd} + (N_{Ed}/N_{Rd})^2$	=	**0.6058**	**< 1.0 (O.K)**

9.8 Design of Tubular Combi-Wall

As per the standard approach (Eurocode EN 1993-5) for assessment and transfer of loads in tubular combi-walls, it assumes that the primary elements (tubular pile) bear the earth pressures and the secondary elements (Z or U-shaped sheeting) bear the water pressure. This approach is based on the standard assumption that the earth pressure will be transferred to the primary elements by virtue of arch forms within the soil.

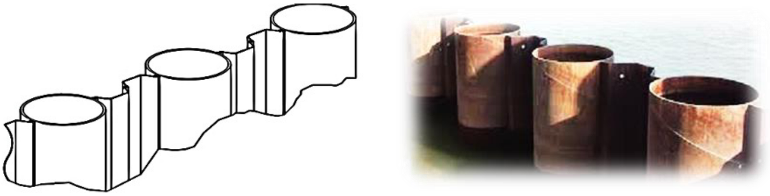


Figure 9.10 View of Combi-pile Wall

Since, the load transfer mechanism of a combi-wall is similar to that of laterally loaded piles; the most important load on wall is bending due to lateral earth pressure. For effective design of combi-wall, large diameter tubular piles are often employed to increase the section modulus of piles. Further, the use of high yield stresses steels make the pile slender or thin walled section. The overall effects of actions due to earth and water pressures shall be determined taking into account the loading on both primary and secondary elements and possible supplementary loading due to arching effects in the ground.

The verification of load resistance due to application of axial load, bending stress, shear stress and circumferential compression, is carried out as outlined in section 9.5. In addition to it, the shell buckling of tubular pile shall also be verified for local shell bending caused by forces from secondary sheet piles.

For the verification of the failure modes related to local buckling, EN 1993-5 refers to EN 1993-1-6, accounting the Strength and Stability of Shell Structures. Apart from the *D/t* ratio, there are several other factors that have an influence on the local buckling behaviour and thereby on the bending moment capacity as listed below:

9.8.1 Local Shell Bending due to Secondary Sheet Piles

The bending strength along vertical axis and local buckling behaviour of tubes with large ovalisation should be examined. Such loading conditions may develop due to transverse loads from secondary sheet piles due to water pressure and also earth pressure in case of soft soil condition, as shown schematically in Figures below. Major part of lateral earth pressures is transferred to the pipe piles directly due to arching effect of soil and water by secondary sheet piles which should be taken into the account to check the circumferential stress.

The mechanism of load transfer from secondary sheet pile to Primary Pipe Pile is shown in **Figure 9.11**.

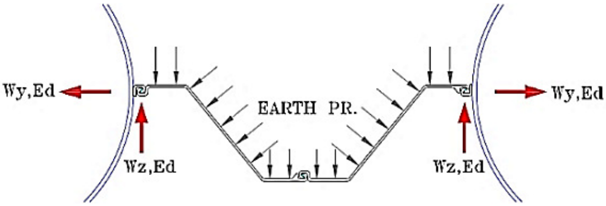


Figure 9.11 Mechanism of Load Transfer from secondary sheet pile to Primary Pipe Pile

The component of force ***Wy*** from secondary sheet pile leads to an ovalisation of the tube. These forces at the interlocks result in a bending of the tubular pile's cross-section also. The empirical co-relations for above case is given in BS EN 1993-5 (2007), Sec-D.2 and Table D-1.

The secondary effect from the resultant forces introduced via the connection to the sheeting may cause to minor ovalisation to the pipe pile, results in an increased radius as shown below.

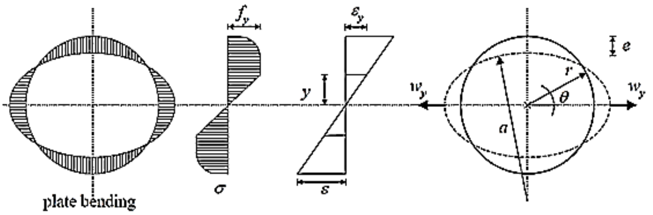


plate bending
f_y
σ
ε_y
y
ε
w_y
w_y
e
r
θ
a

Figure 9.12 Bending of Tubular pile – circumferential and axial bending according to BS EN 1993-1-5

In the design procedure the radius increased by the ovalisation shall be used instead of the initial radius of the geometry 'r`. Also, the effect of the ovalisation of the pipe pile (due to local shell bending) on second moment of area about the vertical axis of wall will cause a reduction. The reduction factor is as given in 9.75. [ref. BS EN 1993-5: 2007; D.2 (4)]

$$\beta_{o,l} = 1 - 1.5\,(e/r) \tag{9.75}$$

In Combi-tube wall, the secondary sheet piles (partition sheet piles) are connected with tubular section through a connector which allows a limited rotation and beyond that it acts like a fixed connection. Therefore, it should be idealised as continuous folded plate structure with fixed support as suggested in BS 1993-5, which is shown below.

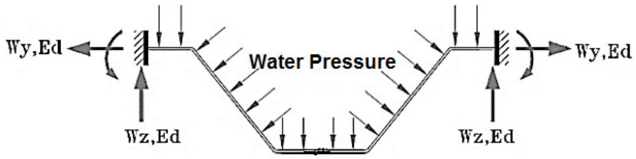

Wy,Ed
Water Pressure
Wy,Ed
Wz,Ed
Wz,Ed

Figure 9.13(a) Idealised secondary sheet piles (Z-piles) as per BS EN 1993-5 for analysis of forces and moments

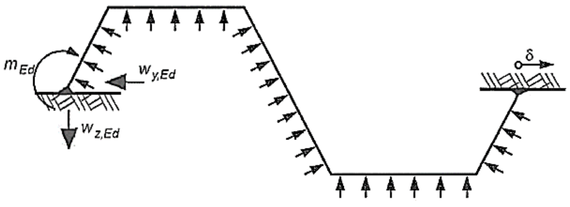

m_{Ed}
$w_{y,Ed}$
$w_{z,Ed}$
δ

Figure 9.13(b) Idealised model of secondary sheet piles (U-type sheet piles)

9.8.2 Axial Membrane Stresses

(due to Axial Forces and Bending Moments)

For combi-wall with class-4 tubular pile sections or extremely slender shells (about *D/t* < 300) where shell buckling is prevailing, pilings are more subjected to plastic bending at failure.

Since, the theoretical buckling stress cannot be reached for real structures, reduction factors should be introduced for the design procedure. In the assessment of against axial compression, critical meridional and circumferential buckling parameters should be considered for assessment of reduction factor which covers the influence of geometrical imperfections (radial deviations and uneven surface) and structural imperfections (e.g. residual stresses) etc. as discussed in section 9.5.

The evaluation of buckling stress, reduction of second moment of area due to ovalisation of pipe and verification of total stress in pile due to overall buckling effect is illustrated in Example-9.2.

Example: 9.2

A thin cell tubular steel pile (class-4 section) having effective length=25m; outer diameter = 1200mm; cell thickness = 16mm; steel grade - 420GP, is installed under a fixed type platform. For ultimate limit state design of tubular pile, the applicable normal load, moment and shear forces are 6150KN, 3150KN.m and 280KN respectively. Verify the load carrying capacity of pile, considering the overall buckling effect in accordance with Eurocode3.

Solution:

Checking of Stresses in Tubular Pile due to Overall Buckling Effect

I) Ultimate Loads on Tubular Pile				
Normal Axial Force [N]	N	=	440.0	T
Normal Shear Force [V]	V	=	1220.0	T
Bending Moment [Mz]	Mz	=	3220.0	t.m

iii) ***Properties of Tubular Pile***				
Eff. Length of Pile	*l*	=	25.0	M
Dia. Of Pile		=	1420.0	mm
Thickness of Shell	t	=	18.0	mm
Corrosion allowance (50 Years)		=	2.0	mm
Net OD of tubular pile	Do	=	1418.0	mm
Net inside dia. of tubular Pile	Di	=	1402.0	mm
Mean Dia.	D	=	1410.0	mm
Mean Radius	r	=	705.0	mm
Net Shell thickness	t	=	16.0	mm
Yield stress	fy	=	420	N/mm^2
Dimensionless length parameter $\omega = l/\sqrt{r.t}$	ω	=	235.39	
Area of C/S of pile	Ax	=	35437.17	mm^2
Iyy = Izz		=	8.81E+09	mm^4
Sy = Izz / y		=	9368801	mm^3
Axial Stress	σ_{x1}	=	17.21	N/mm^2
Bending Stress	σ_{x2}	=	128.89	N/mm^2
Axial Stress	$\sigma_{x,Ed}$	=	146.10	N/mm^2
Shear Stress $\tau_{x\theta,Ed} = (V / \pi.r.t)$	$\tau_{x\theta,Ed}$	=	34.43	N/mm^2

A) Check for Stresses & Buckling Effect in Tubular Pile due to Secondary Element (Ovalisation of Tubular Pile due to Loads from Secondary Sheet Piles)

Mean Radius of Shell (after corrosion loss)	**r**	=	705.00	mm
Thickness of Shell (after loss due to corrosion in 50 years in submerged condition)	**t**	=	16.0	mm
Modulus of Elasticity	**E**	=	210000	N/mm^2

Yield Stress of Steel (Tubular Pile)	**fy**	=	420	N/mm^2
EI = E. t^3 / 12 (Stiffness of Shell Bending ; [BS 1993-5 #D.2.1 (5))	**EI**	=	7.17E+07	N.mm^2
Support reaction/m length from Sec. sheet pile [BS 1993-5# Cl. 5.5.2]	$W_{y,Ed}$	=	46.00	KN/m
Local moment /m length (from Sec. sheet pile)	m_{Ed}	=	22.10	kN.m/m
Ovalisation due to local bending = $0.068W_{y,ED}$*(r^3/ EI) < 0.1* r [BS 1993-5 # D.2.1 (5)]	**e**	=	15.29	mm < 0.1r
Radius of curvature at ovalisation (a); [BS 1993-5 #D.2.1 (5)] $a = \frac{r}{1-\frac{3e}{r}}$	**a**	=	754.066	mm
t/a > 1/60	**t/a**	=	0.021	-
$\beta_{ol} = 1 - 1.5\,(e/r)$ [BS 1993-5 #D.2.1 (4)]	β_{ol}	=	0.967	-
= 0.137 m_{Ed} + 0.182 W_{yEd} * r [BS 1993-5 #Table D-1]	M_{Ed1}	=	8.93	kN.m/m
= -0.5 m_{Ed} - 0.318 W_{yEd} * r [BS 1993-5 #Table D-1]	M_{Ed2}	=	-21.36	kN.m/m
= 0.637 m_{Ed}/r + 0.50 W_{yEd} [BS 1993-5 #Table D-1]	N_{Ed1}	=	42.97	KN/m
= 0 [BS 1993-5 #Table D-1]	N_{Ed2}	=	0.00	KN/m
= 0 [BS 1993-5 #Table D-1]	V_A	=	0.00	KN/m

$= -0.637\, m_{Ed} / r - 0.50\, W_{yEd}$ [BS 1993-5 #Table D-1]	V_B	=	-42.97	KN/m
$= 0.1488\, W_{yEd} * r^3 / EI$ [BS 1993-5 #Table D-1]	ΔD_{BD}	=	33.46	mm
$= -0.1368\, W_{yEd} * r^3 / EI$ [BS 1993-5 #Table D-1]	ΔD_{AC}	=	-30.762	mm
$r + \Delta D_{BD}$		=	738.46	mm
Plastic moment Resistance capacity of circular Shell section $M_{Rk} = 4.r^2.\, t\, .\, (f_y / \gamma_{Mo})$	M_{Rk}	=	14658.30	kN.m/m
Design plastic Resistance to normal force of circular Shell section $N_{Rk} = \pi\, .r\, .\, t\, .\, (f_y / \gamma_{Mo})$	N_{Rk}	=	1559.00	kN/m
	χ	=	1.166	
$\frac{N_{Ed}}{\frac{\chi N_{Rk}}{\gamma_{M1}}}$		=	0.026	
$1.5 \frac{M_{Ed}}{\frac{M_{Rk}}{\gamma_{M1}}}$		=	0.002	
$\frac{N_{Ed}}{\frac{\chi N_{Rk}}{\gamma_{M1}}} + 1.5 \frac{M_{Ed}}{\frac{M_{Rk}}{\gamma_{M1}}}$ [BS 1993-5 # D-2.2 (12)]		=	0.0284	< 1 (O.K)
Due to maximum lateral pressure transferred by secondary sheet pile to combi tube, maximum elongating is 4.75% and ratio of forces are negligible, Hence it is safe in case of local buckling due to interaction of Pipe Pile and double AZ-infill sheet piles.				

B) Verification of Buckling against Axial Compression

After ovalisation, Mean radius of cell (r = r + Δ_{BD})	r	=	738.46	mm
Net cell thickness (for design)	t	=	16.0	mm
Modulus of Elasticity	E	=	210000	N/mm^2
Yield Stress of Steel (Tubular Pile)	fy		420	N/mm^2
r/t		=	46.15	
0.03 E/fy		=	15.00	
r/t > 0.03 E/fy - Axial Buckling check is required				
	ω	=	235.39	
	ω > 0.5*r/t		Long Cylinder	
Fixidity Condition of Pile				
Top End BC-1 (Hinged) & Bottom BC-2 (fixed)]	C_{xb}	=	3	
$1+\frac{0.2}{C_{xb}}\left[1-2\,\omega.{}^{t}/_{r}\right] \ngtr 0.6$	C_x	=	0.387	-
0.6 . E . Cx . t/r	$\sigma_{x,Rcr}$	=	1055.6	N/mm^2
$\bar{\lambda}_x = \sqrt{f_y/\sigma_{x,Rcr}}$	λ_x	=	0.631	
Manufacturing Tolerance Quality Parameter	Q	=	25	
$\Delta w_k = \frac{1}{Q}\sqrt{\frac{r}{t}}.t$	Δw_k	=	4.348	
$\alpha_x = \frac{0.62}{1+1.91(\Delta w_k/t)^{1.44}}$	α_x	=	0.480	
	λo	=	0.2	
	β	=	0.6	
	η	=	1	

$\bar{\lambda}_p = \sqrt{\frac{\alpha_x}{1-\beta}}$	$\bar{\lambda}_p$	=	1.095	
$\bar{\lambda}_0 < \bar{\lambda}_x < \bar{\lambda}_p$				
$\chi_x = 1-\beta\left(\frac{\bar{\lambda}-\bar{\lambda}_0}{\bar{\lambda}_p-\bar{\lambda}_0}\right)^{\eta}$	χ_x	=	0.711	
	γ_{M1}	=	1.10	
$\sigma_{x,Rd} = \chi_x \cdot f_y/\gamma_{M1}$	**$\sigma_{x,Rd}$**	=	**271.56**	**N/mm^2**
$\sigma_{x,Ed} < \sigma_{x,Rd}$				

C) Verification of Buckling against Shear Forces

V / π.r.t	$\tau_{x\theta,Ed}$	=	34.43	N/mm^2
$0.16\left(\frac{E}{f_y}\right)^{0.67}$		=	10.29	
$\frac{r}{t} < 0.16\left(\frac{E}{f_y}\right)^{0.67}$ Buckling check against shear force is required				
8.7 * r/t		=	401.54	
10 < ω < 8.7 r/t => Med. Length cylinder				
	C_τ	=	1.000	
$\tau_{x\theta,Rcr} = 0.75\,E\,C_\tau\sqrt{\frac{1}{\omega}}(t/r)$	$\tau_{x\theta,Rcr}$	=	222.42	N/mm^2
$\bar{\lambda}_\tau = \sqrt{(f_y/\sqrt{3})/\tau_{x\theta,Rcr}}$	$\bar{\lambda}_\tau$	=	1.04	
Shear elastic imperfection factor (Class-B)	α_τ	=	0.65	
	$\bar{\lambda}_{\tau 0}$	=	0.40	
	β	=	0.60	
	η	=	1.0	

Plastic limit of relative slenderness $\bar{\lambda}_p = \sqrt{\dfrac{\alpha_\tau}{1-\beta}}$	$\bar{\lambda}_p$	=	1.275	
$\bar{\lambda}_0 < \bar{\lambda}_x < \bar{\lambda}_p$				
$\chi_\tau = 1 - \beta\left(\dfrac{\bar{\lambda}_\tau - \bar{\lambda}_0}{\bar{\lambda}_p - \bar{\lambda}_0}\right)^\eta$	χ_τ	=	0.558	
$\sigma_{x\theta,Rd} = \dfrac{\chi_\tau \ . f_y/\sqrt{3}}{\gamma_{M1}}$	$\sigma_{x\theta,Rd}$	=	123.05	N/mm^2
$\tau_{x\theta,Ed} \ < \ \tau_{x\theta,Rd}$				

D) Verification of Buckling against Circumferential Compression

Circumferential pressure is due to differential water pressure;

Differential water head in tubular piles	ΔH_w	=	2.50	M
Differential water Pressure	ΔP_w	=	0.025	N/mm^2
$\sigma_{\theta,Ed} = \Delta P_W \ . \ r/t$	$\boldsymbol{\sigma_{\theta,Ed}}$	=	1.17	N/mm^2
	C_θ	=	1.00	
1.63*r/t		=	75.23	
	$\dfrac{\omega}{C_\theta}$	=	235.39	
$\dfrac{\omega}{C_\theta} > 1.63 \ r/t$; Hence, Long Cylinder				
$E\left(\dfrac{t}{r}\right)^2\left[0.275 + 2.03\left(\dfrac{C_\theta . r}{\omega \ . \ t}\right)^4\right]$	$\sigma_{\theta,Rcr}$	=	27.41	N/mm^2
$\bar{\lambda}_\theta = \sqrt{f_y/\sigma_{\theta,Rcr}}$	$\bar{\lambda}_\theta$	=	3.91	
For Class-B section	α_θ	=	0.65	
	$\bar{\lambda}_0$	=	0.40	
	β	=	0.60	
	η	=	1.00	

$\bar{\lambda}_p = \sqrt{\frac{\alpha_\theta}{1 - \beta}}$	$\bar{\lambda}_p$	=	1.27	
$\chi_\theta = \frac{\alpha_\theta}{\bar{\lambda}_\theta}$	χ_θ	=	0.510	
$\sigma_{\theta,Rd} = \chi_\theta \cdot f_y/\gamma_{M1}$	$\sigma_{\theta,Rd}$	=	194.69	N/mm^2
$\sigma_{\theta,Ed} \leq \sigma_{\theta,Rd}$				
$K_x = 1.25 + 0.75\chi_x$	K_x	=	1.783	
$K_\theta = 1.25 + 0.75\chi_\theta$	K_θ	=	1.632	
$K_\tau = 1.25 + 0.55\chi_\tau$	K_τ	=	1.890	
$K_i = \left(\chi_x \cdot \chi_\theta\right)^2$	K_i	=	0.36	
$\left(\frac{\sigma_{x,Ed}}{\sigma_{x,Rd}}\right)^{k_x}$		=	0.33	
$K_i\left(\frac{\sigma_{x,Ed}}{\sigma_{x,Rd}}\right)\left(\frac{\sigma_{\theta,Ed}}{\sigma_{\theta,Rd}}\right)$		=	-1.17E-03	
$\left(\frac{\sigma_{\theta,Ed}}{\sigma_{\theta,Rd}}\right)^{k_\theta}$		=	2.35E-04	
$\left(\frac{\tau_{x\theta,Ed}}{\tau_{x\theta,Rd}}\right)^{k_\tau}$		=	9.01E-02	
$\left(\frac{\sigma_{x,Ed}}{\sigma_{x,Rd}}\right)^{k_x} - K_i\left(\frac{\sigma_{x,Ed}}{\sigma_{x,Rd}}\right)\left(\frac{\sigma_{\theta,Ed}}{\sigma_{\theta,Rd}}\right) + \left(\frac{\sigma_{\theta,Ed}}{\sigma_{\theta,Rd}}\right)^{k_\theta} + \left(\frac{\tau_{x\theta,Ed}}{\tau_{x\theta,Rd}}\right)^{k_\tau}$		=	**0.42**	**< 1** **(O.K.)**

9.9 Design of Tie Rods

A majority of failures of anchored walls occur in the tie rods and anchors. For designing of tie rods, following factors should be considered:

a) Tensile resistance of anchor bar or tie-rod
b) Elongation of tie bar under load
c) Settlement effect of the fill upon the tie rod.
d) Corrosion allowance for the tie bar

9.9.1 Tensile Resistance

The tie rods are suggested to be designed for a load 20 % greater than the allowable load. When the tie rods are inclined to the plane of the wall, the design tie rod force will be equal to the Ta / cos(α)

Where, Ta = Design tie rod pull after 20% increase

α = angle of tie rod inclination

Tie rod inclination further induces axial force in the sheet piling given by *Ta x* tan (α). The axial component of inclined anchor force and any external axial loads are assumed to be resisted by the wall.

The tensile resistance of a tie bar depends upon the cross-sectional area of the bar and the cross-sectional area of the thread or connecting part to the structure. BS EN1993-5 specifies the tensile resistance of tie-bars shall be adopted as the lesser of the tensile resistance of the thread or the tensile resistance of the shaft, at any time during the life of the structure. For shaft area (As), the tensile resistance (T_a) of anchor shall be;

$$T_a = 0.6 \times f_y \times A_s \tag{9.76}$$

For threaded area (At), the tensile resistance (T_t) of anchor shall be;

$$\mathrm{T_t = K_t \times 0.6 \times f_y \times A_t} \tag{9.77}$$

The effect of the K_t reduction factor applied on the tensile resistance of the threads to allow for the possibility of bending when settlement occurs in the fill.

To allow bending and tension in the thread of anchor $\mathrm{K_t}$ is recommended as 0.6 at the connections. If structural detailing eliminates bending at connection, K_t shall be adopted as 0.90. To economise the design of tie-rods, use of upset bars allows the cross-sectional area of the thread to be greater than the shaft. Particularly in seismic areas, it is recommended that failure should occur in the shaft first, and not the threads, as this will allow greater and more uniform deformation to occur before failure.

9.9.2 Grades of Steel for Tie-rods

The most common steel grades of material available for tie-rods include 355, 460, 500 and 700 grades (which typically have yield valuesf_y, of 355, 460, 500 & 700 N/mm^2 respectively). The choice of steel grade depends on a number of factors, whilst the higher strength steel will always produce the anchor of lightest weight. However, this may not be suitable for stiffness requirements, onsite welding or lead-times. It is recommended that high strength steel grades (f_y more than 500 N/mm^2) shall be used with caution.

9.9.3 Elongation in Tie Rods

Elongation in tie rods under the design load should be checked, this should be based on the shaft diameter of the tie bar. Use of higher grade of steel bars economise the tie-rod section, but reduced cross-sectional area increases the elongation in bars, which shall be checked with permissible deflection in wall. The increased stiffness of the threaded area (if larger) can be neglected as tie bars are typically sufficiently long for the effect to be negligible.

9.9.4 Settlement Effect

If settlement is expected in the fill, it is good practice to minimise the effect upon the tie bar by using articulated joints at appropriate locations and allowing the bar to rotate with settlement. Settlement ducts are sometimes used as a means of reducing the effect of settlement on the tie bar. Tie bars are placed inside large (typically 200-350mm) plastic tubes, the bar lies on the bottom of the tube and as fill settles the tube settles leaving the bar suspended in the tube.

9.9.5 Corrosion Protection

In aggressive environment, steel tie-rods are susceptible to corrosion and it is important to allow for steel loss in the tie bar for anticipated corrosion. Consideration should be given to the probable corrosion rates and consequential loss of tie bar section depending on their position in the structure as outlined in Chapter - 8.

9.9.6 Design of Wales

Wales which transfer the tie rod forces to the sheet piling are usually composed of back-to-back channels as explained in chapter-7. The wale is assumed to act as a continuous flexural member over simple supports at the tie rod locations. The maximum bending moment in the wale may be approximated as;

$$M_{max} = T_u \times S^2/10$$

Where, T_u = anchor force per meter length of wall

S = distance between adjacent tie rods Sizing of the wale cross section, wale-to-piling connections,

For sizing purposes, the response of a wale may be assumed to be somewhere between that of a continuous beam on several supports (the tie rods) and a single span on simple supports. Therefore, the maximum bending moment for design of continuous segment will be;

$$M_{max} = T_u \times S^2/10$$

And at end spans will be;

$$M_{max} = T_u \times S^2/\ 8$$

The design should also take into consideration of web crippling and possible torsion, biaxial bending, and shear produced by inclined tie rods.

Example: 9.5

*A 38m long anchored sheet pile wall for waterfront structure is installed in stratified soil stratum as shown in figure E-9.1. The installation details of wall are given below and characteristic details of backfill as well as founding strata are given in **Table: E-9.1**.*

Lowest water level on waterfront side is 0.0m Level.

Design Inputs (anchored Retaining Wall Wall)

Finished level of Backfill / Wall head	=	+ 6.0	m Lvl.
Dredge Level / Mud level	=	-18.0	m Lvl.
Founding Level of	=	-32.0	m Lvl.
Total depth of Wall	=	38.0	M
Design Ground Water Table on backfill side	=	+ 2.0	m Lvl.
Lowest Low Water Level on waterfront side	=	± 0.0	m Lvl.
Design Live Load Surcharge on backfill (q)	=	25.0	KN/m^2
φ of Backfill & Founding material	=	32	deg.
Unit wt. of soil (γ')	=	18.0	KN/m^3
Unit wt. of concrete	=	24.0	KN/m^3

***Table E: 9.1 -** Properties of Soil Strata*

Soil Layer	Levels of soil strata below finished backfill	Soil type	Density (γ) (KN/m^3)	Cohesion (c) (KN/m^2)	Angle of friction (ϕ)
1	+ 6.0 m to -9.0 m	Silty sand	17.0	0.0	32 deg
2	-9.0 m to -12.0 m	Med. Dense Sand	18.0	0.0	33 deg
4	-12.0 m to -18.0 m	Very Stiff Clay	21.0	100.0	0 deg
5	-18.0 m to -32.0m	Dense Sand	20.0	0.0	35 deg

Analyse the retaining structure to:- a) identify the forces, b) design the structural section of main wall, anchor wall and tie-rods c) verify the overall stability of the system in normal condition, excluding the seismic case.

Solution:

A. Estimation of Loads and Structural analysis of Wall

i) *General arrangement of wall*

For assessment of loads and internal forces using soil-structure-interaction analysis, the profile of wall and stratified soil is shown below in **Figure E-9.1**.

Profile of Soil at Retaining Wall

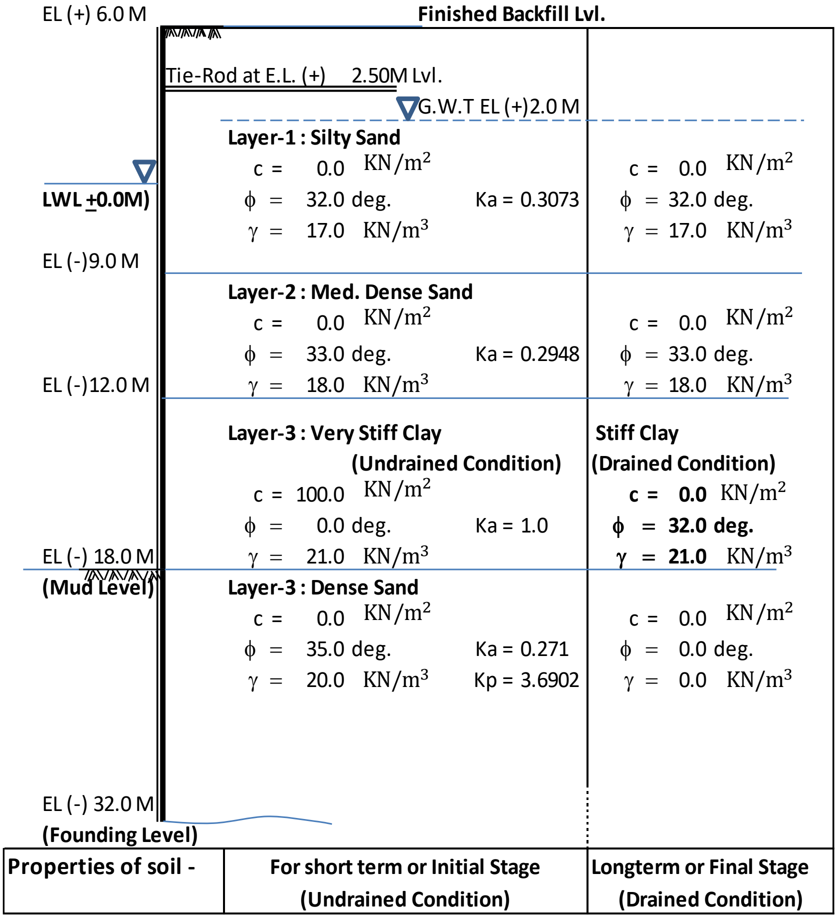

EL (+) 6.0 M
Finished Backfill Lvl.
Tie-Rod at E.L. (+) 2.50M Lvl.
G.W.T EL (+)2.0 M
Layer-1 : Silty Sand
c = 0.0 KN/m²
ϕ = 32.0 deg.
Ka = 0.3073
γ = 17.0 KN/m³
c = 0.0 KN/m²
ϕ = 32.0 deg.
γ = 17.0 KN/m³
LWL +0.0M)
EL (-)9.0 M
Layer-2 : Med. Dense Sand
c = 0.0 KN/m²
ϕ = 33.0 deg.
Ka = 0.2948
γ = 18.0 KN/m³
c = 0.0 KN/m²
ϕ = 33.0 deg.
γ = 18.0 KN/m³
EL (-)12.0 M
Layer-3 : Very Stiff Clay
(Undrained Condition)
c = 100.0 KN/m²
ϕ = 0.0 deg.
Ka = 1.0
γ = 21.0 KN/m³
Stiff Clay
(Drained Condition)
c = 0.0 KN/m²
ϕ = 32.0 deg.
γ = 21.0 KN/m³
EL (-) 18.0 M
(Mud Level)
Layer-3 : Dense Sand
c = 0.0 KN/m²
ϕ = 35.0 deg.
Ka = 0.271
γ = 20.0 KN/m³
Kp = 3.6902
c = 0.0 KN/m²
ϕ = 0.0 deg.
γ = 0.0 KN/m³
EL (-) 32.0 M
(Founding Level)
Properties of soil -
For short term or Initial Stage (Undrained Condition)
Longterm or Final Stage (Drained Condition)

Figure E : 9.1 - Profile of soil and wall for soil

In case of anchored wall, first and foremost requirement is placement of anchor in non-yielding soil zone. It should be located outside the potential active failure zone or in passive zone which is generally obtained by drawing a slip-line based on Mohr's rupture envelope at a slope of (45 + φ/2) behind the wall and further passive zone is identified as shown in **Figure E-9.2**.

Based on above graphical analysis, the anchor (sheet pile) wall is placed at a distance of 40m behind the wall as shown in **Figure E-9.2** below.

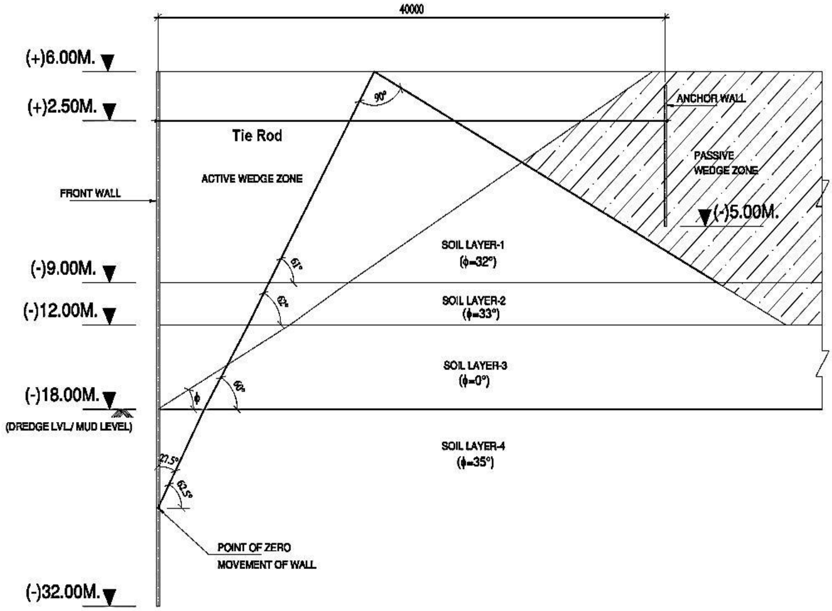

40000
(+)6.00M.
(+)2.50M.
Tie Rod
ACTIVE WEDGE ZONE
FRONT WALL
90°
ANCHOR WALL
PASSIVE WEDGE ZONE
(-)5.00M.
(-)9.00M.
(-)12.00M.
(-)18.00M.
(DREDGE LVL/ MUD LEVEL)
61°
62°
60°
ϕ
SOIL LAYER-1 (ϕ=32°)
SOIL LAYER-2 (ϕ=33°)
SOIL LAYER-3 (ϕ=0°)
SOIL LAYER-4 (ϕ=35°)
27.5°
62.5°
POINT OF ZERO MOVEMENT OF WALL
(-)32.00M.

Figure E:9-2 Location of anchorage zone behind the anchored retaining wall

ii) ***Calculation of Lateral Loads applicable on wall***

Earth Pressure (Short term / at initial stage or in undrained condition)

This can be considered as a case of a smooth vertical wall and horizontally retained soil surface ($\delta = 0$, $\alpha = 90°$, and $\beta=0$),Therefore, Using Rankine's formula for earth pressure;

Lateral (Active) Earth Pressures

$\sigma a\ (+6) = Ka.\ \gamma'.H + Ka.q \quad = \quad 7.7\ \text{KN/m}^2$

$\sigma a\ (+2) = Ka.\ \gamma'.H + Ka.q \quad = \quad 28.6\ \text{KN/m}^2$

$\sigma a\ (-9) = Ka.\ \gamma'.H + Ka.q \quad = \quad 52.2 \quad KN/m^2$

$\sigma a\ (-12) = Ka.\ \gamma'.H + Ka.q \quad = \quad 57.2 \quad KN/m^2$

$\sigma a\ (-12) = Ka.\ \gamma'.H + Ka.q - 2c\ Ka \quad = \quad 0.0 \quad KN/m^2$

$\sigma a\ (-18) = Ka.\ \gamma'.H + Ka.q - 2c\ Ka \quad = \quad 60.0 \quad KN/m^2$

Hydrostatic Pressures

$\sigma h \quad = \gamma_w\ .\ H \quad = \quad 20.0 \quad KN/m^2$

Case-I : *Lateral Pressures on Structure for Analysis (Initial Stage)*

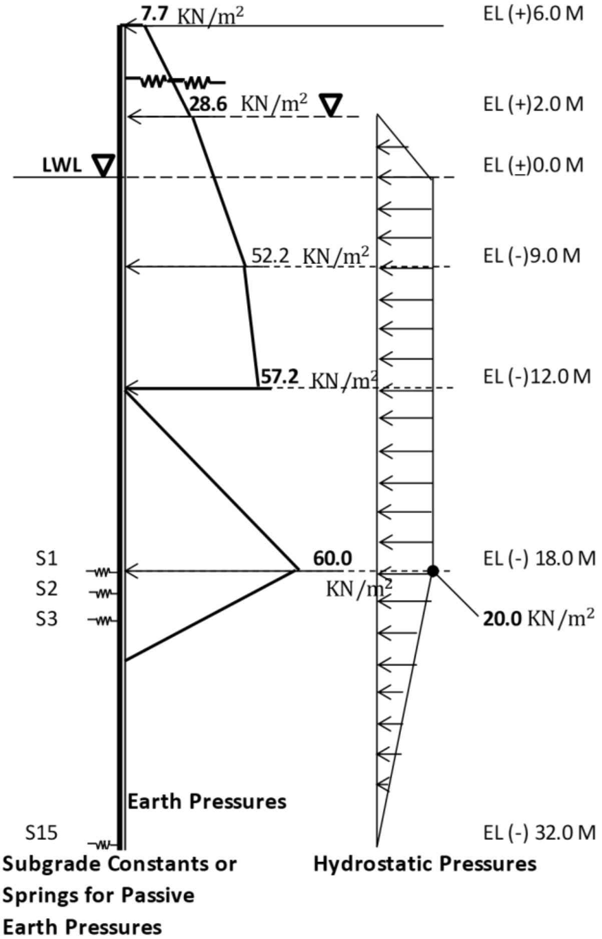

7.7 KN/m²
EL (+)6.0 M
28.6 KN/m²
EL (+)2.0 M
LWL
EL (±)0.0 M
52.2 KN/m²
EL (-)9.0 M
57.2 KN/m²
EL (-)12.0 M
S1
S2
S3
60.0 KN/m²
EL (-) 18.0 M
20.0 KN/m²
Earth Pressures
S15
EL (-) 32.0 M
Subgrade Constants or Springs for Passive Earth Pressures
Hydrostatic Pressures

Figure E: 9.3 Lateral Earth Pressure diagram (at initial stage or in undrained condition) and hydrostatic pressure diagram

The properties of cohesionless soil will remain same in undrained condition; however, in cohesive soil it will change for long term (drained condition) due to strain softening after mobilisation of soil behind the wall. Referring Figure-4.4 (Chapter-4); the design shear value of low plasticity cohesive soil will have;

- drained friction angle as 32 degree, and
- cohesion value will be zero (NIL).

Accordingly, the lateral earth pressure applicable on wall in drained condition will be determined as shown in Figure E-9.5.

Case-II : Lateral Pressures on Structure for Analysis (Final Stage)

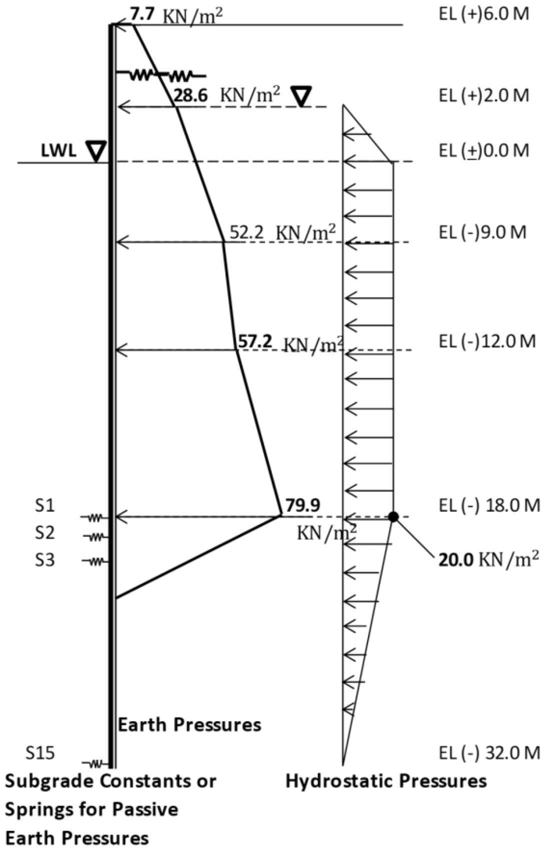

7.7 KN/m²
EL (+)6.0 M
28.6 KN/m²
EL (+)2.0 M
LWL
EL (±)0.0 M
52.2 KN/m²
EL (-)9.0 M
57.2 KN/m²
EL (-)12.0 M
S1
S2
S3
79.9 KN/m²
EL (-) 18.0 M
20.0 KN/m²
Earth Pressures
S15
EL (-) 32.0 M
Subgrade Constants or Springs for Passive Earth Pressures
Hydrostatic Pressures

Figure E: 9.4 Lateral Earth Pressure diagram (at final stage or in drained condition) and hydrostatic pressure diagram

iii) Virtual Geometrical model of wall and estimation of soil subgrade modulus

For analysis of wall with reasonable accuracy, the virtual geometric model of sheet pile wall is prepared using soil-structure interaction. There are several strategies to capture the interaction between the soil and the structure. Here, a very simple approach, known as the Beam-on-Elastic-Foundation, using Winkler's –Support Model is used. A series of distributed independent elastic soil springs at 1.0 m intervals below dredge level are generated along the soil support and connected to structural elements.

Soil subgrade modulus

The soil subgrade modulus along various depth of wall is estimated using API model for cohesionless soil correspond to 25mm deflection (y_{25}) as given below.

Calculation of Soil-subgrade modulus using API model

H (m)	**A**	**Pu** (KPa)	$\frac{K.H}{A.P_u}$	$\mathbf{y_1}$ (m)	$\mathbf{P_1}$ (Kpa)	**K = p_1/y_1** (KN/m^2/m)
0	3.00	0.0	0.00	0.025	0	0
1	2.20	58.5	170.94	0.025	129	5146
2	1.40	171.0	183.79	0.025	239	9574
3	0.90	337.5	217.28	0.025	304	12150
4	0.90	558.0	175.23	0.025	502	20082
5	0.90	832.5	146.81	0.025	748	29931
6	0.90	1161.0	126.33	0.025	1041	41645
7	0.90	1543.5	110.86	0.025	1378	55133
8	0.90	1980.0	98.77	0.025	1757	70266
9	0.90	2470.5	89.05	0.025	2172	86890
10	0.90	3015.0	81.08	0.025	2621	104837
11	0.90	3613.5	74.41	0.025	3098	123934
12	0.90	4266.0	68.76	0.025	3600	144015
13	0.90	4972.5	63.91	0.025	4123	164925
14	0.90	5733.0	59.69	0.025	4663	186524
15	0.90	3712.5	98.77	0.025	3294	131748

The effective normal stiffness *k of tie-rod* (in kN/m per m run) is calculated as; $k = \frac{A.E}{L}$

For, 95mm dia. steel tie rod @ 2.127M spacing, the value of 'k' = 17495 KN/m per m run.

Applying lateral earth pressures and hydrostatic pressures as factored static load with load factor for structural analysis, the ultimate design loads observed on wall is presented below for design.

iv) ***Design Loads for main wall elements (H-Piles)***

Ultimate Shear Force in retaining wall = 829.19 KN/m

Tensile force in Tie-rod (unfactored) = 610 KN/m

For deep retaining wall, a high modulus combined section (combination of H-beams and Z-sheet piles) are considered. The system width of H-Z sheet pile section is 2.127m and corresponding design loads are indicated below based on computer model analysis results.

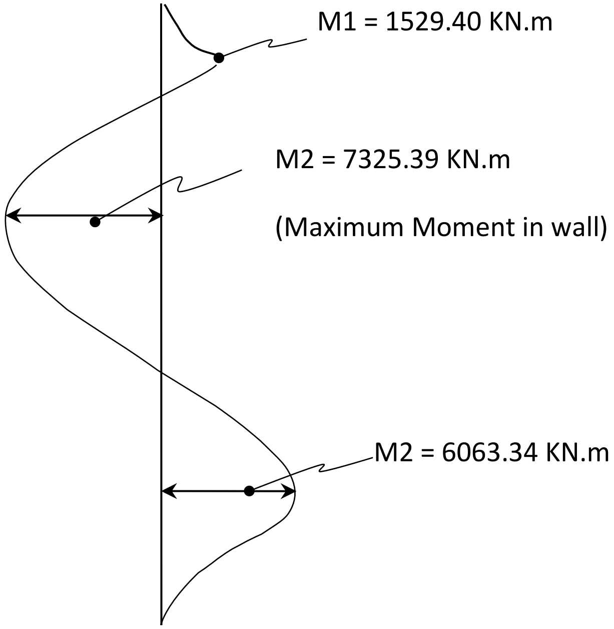


Figure E: 9.5 Lateral Earth Pressure diagram (at final stage or in drained condition)

B. Structural Design of main wall element (H section)

i) Design Loads

Design compressive force (N_{ED}) = **30.0** KN

(Loads due to capping beam etc. on wall)

Bending Moment due to lateral load(M) = **6104.5** KNm

Design bending moment (M_{Ed}) = **7325.39** KN.m

Design shear force (V_{Ed}) = **1763.7** KN

Reduction factor for yield stress of steel due to differential water pressure ρ_{ρ} = **1.0**

Design Yield stress of steel($\rho_{\rho} * f_y$) = **420.0** N/mm^2

Young's Modulus (Es) of pile material = **210000** N/mm^2

Partial material factor (γ_{M0}) = **1.00**

Design life considered for wall = **50.00** years

ii) Section properties of sheet pile wall element

For protection of pile in splash zone, concrete lining is considered and in submerged zone, sacrificial thickness is provided as corrosion allowance. The summary of design loads and design section properties of H-piles are given below.

Reduction in sheet pile sections due to corrosion loss :

Active face of sheet pile in splash zone is encased in concrete

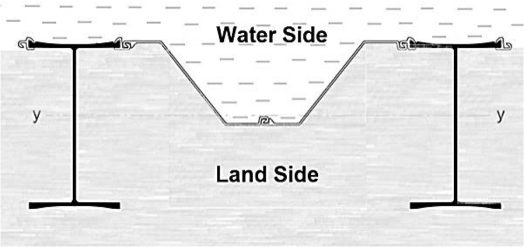
Active face of sheet piling (in splash zone) = 0.00 mm

Active face of sheet piling (submerged zone) = 1.75 mm

Passive face of sheet piling = 0.60 mm

Sheet Pile Section adopted = **HZ- 1180**

System width of H-Z piles = 2.127 m



Pile section properties :			Full properties	Reduced properties (allowing for corrosion)
Pile type		=	**H**	H mm
Pile System width	B	=	**2160** mm	**2160** mm
Width of flange	b	=	**460** mm	**460** mm
Flange thickness	t_f	=	**37** mm	**37.00** mm (Sea side) **37.00** mm (Land side)
Web thickness	t_w	=	**22** mm	**22.00** mm
Overall height of section	h	=	**1180** mm	**1180** mm
Moment of Inertia	Iy	=	**1E+06** cm^4	**1094540** cm^4
Moment of Inertia	Iz	=	**54720** cm^4	**54720** cm^4
Plastic section modulus	W_{pl}	=	**-**	**-**
Elastic section modulus	W_{el}	=	**20205** cm^3	**20205** cm^3
Cross section	A	=	**545.0** cm^2	**545.0** cm^2
	EI	=	**2E+06** $KN.m^2$	**2298534** $KN.m^2$

iii) Shear Buckling of Web in H- pile

To satisfy the criteria unstiffned web buckling $\boldsymbol{h/t_w \leq 72.\varepsilon/\eta}$

h = **1180** mm

t_w = **22.00** mm

$\varepsilon = \sqrt{235/f_y}$ ε = **0.75**

η = **1.2**

h / t_w = **53.64**

$72\varepsilon / \eta$ = **44.88**

for, $h / t_w > 72\,\varepsilon / \eta$ **web shear buckling shall be varified**

iv) Buckling of flange in H- pile

To satisfy the criteria unstiffned flange buckling $b/t_f \le 14\,\varepsilon$

b = **460** mm

t_w = **22.00** mm

Flange outstand $c = (b - t_w)/2$ = **219** mm

(neglecting the fillet radius)

t_f = **37.00** mm

c/ t_f = **5.92**

14ε = **10.47**

for, $c/ t_f < 14\varepsilon$ **Flange Outstand is satisfactory**

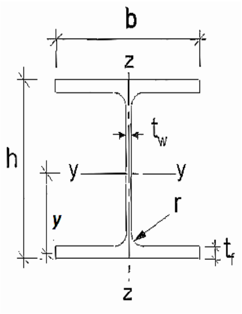
v) Shear Capacity of section

For class-3 sections, maximum shear stress in beam should satisfy

Shear stress $\tau_{Ed} = V_{Ed}.S / I_y.t_f$; $$\frac{\tau_{Ed}}{f_y/[\sqrt{3}\gamma_{M0}]} \le 1.0$$

Design shear force in wall = **1763.7** KN

Design shear force in H- pile (V_{Ed}) = **1763.7** KN



b = **460** mm

t_f = **37.00** mm

$y = h/2$ = **590.0** mm

$A = b_f \, . \, t_f$ = **17020** mm^2

$S = A.y$ = **1E+07** mm^3

I_y = **1094540** cm^4

$\tau_{Ed} = V_{Ed}.S / I_y.t_f$; = **43.73** N /mm^2

$f_y/\sqrt{3}\ \gamma_{M0}$ = **242.49** N /mm^2

τ_{Ed} / $f_y/\sqrt{3}\ \gamma_{M0}$ = **0.18** < 1.0 **O.K**

vi) Moment Resistance Capacity

Section modulus of H- Pile (W) = **20205** cm^3

Applied moment in wall = **7325.40** KN.m

Design bending moment in H-pile(M_{Ed}) = **7325.40** KN.m

$M_{cRd} = W_{el}\, {}^{f_y}/_{\gamma_{M0}}$ = **8486.10** KN.m

M_{Ed} / M_{cRd} = **0.863** **Moment capacity satisfactory**

vii)Check for Axial Compression

Applied compressive force in wall = **30.0** KN

Design compressive force in Pile (N_{ED}) = **30.0** KN

Area of cross-section of H-pile (A) = **545.0** cm^2

$N_{cRd} = A\, f_y / \gamma_{M0}$ = **1635** KN

$N_{Ed}/N_{cRd} < 1.0$ N_{Ed}/N_{cRd} = **0.0183** **Axial capacity satisfactory**

viii) Check for axial Buckling

Wall type = **Propped**

Unsupported length (L) = **24.0** m

Effective buckling length factor = **1.0**

Effective buckling length (l) = **24.0** m

$N_{cr} = E\, I\, \beta_D\, \pi^2 / l^2$ N_{cr} = **39385** KN

$N_{Ed}/N_{cr} < 0.04$ N_{ED}/N_{cr} = **0.0008**

$\lambda_1 = 93.3\, \varepsilon$ = **69.79** radius of gyration (i)of H-pile

about y-y axis $i = \sqrt{I_y/_A}$ or i = **44.81** cm

L_{cr} = **24.00** m

$\bar{\lambda} = \frac{L_{cr}}{i}\frac{1}{\lambda_1}$ $\bar{\lambda}$ = **0.77**

$\bar{\lambda}$ > **0.20** **Buckling resistance needs to be verified.**

reduction factor, χ = **0.956**

buckling resistance in compression

$N_{b,Rd} = \frac{\chi A f_y}{\gamma_{M1}}$ = **21883** KN

$N_{Ed}/N_{b,Rd}$ = **0.001** < 1.0

Section is satisfactory in axial buckling

ix) *Combined Bending and Compression*

Longitudinal stress $\sigma_{x,Ed}$ in combined bending moment and axial force must satisfy the yield criterion $\sigma_{xEd} \leq f_y/\gamma_{M0}$

f_y/γ_{M0} = **420.00** N/mm^2

Based on elastic theory,

$$\sigma_{xEd} = {}^{N_{Ed}}/_{A} + M_{zEd}.z/I_y$$

N_{Ed} = **30.00** KN

A = **545.00** cm^2

${}^{N_{Ed}}/_{A}$ = **0.55** N/mm^2

M_{zEd} = **7325.40** KN .m

I_y = **1094540** cm^4

z = h/2 = **590.00** mm

$M_{zEd}.z/I_y$ = **394.87** N/mm^2

σ_{xEd} = **395.42** N/mm^2

$\sigma_{xEd}/f_y/\gamma_{M0}$ = **0.941** < **1.0 (O.K)**

x) ***Combined Stress due to Bending and Shear***

Design resistance in combined bending moment and shear force

$$\sigma_v = \left[\sigma_{xEd}^{\ 2} + \ 3.\tau_{Ed}^{\ 2}\right]^{.2} \le {}^{f_y}/_{\gamma_{M0}}$$

σ_{xEd} = **395.42** N/mm^2

τ_{Ed} = **43.73** N /mm^2

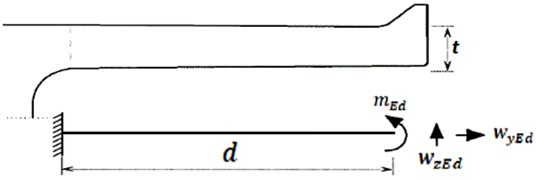
σ_v = **402.61** N /mm^2

f_y/γ_{M0} = **420.00** N /mm^2

$\sigma_v/f_y/\gamma_{M0}$ = **0.96** < **1.0 (O.K)**

xi) ***Verification of Local Plate Bending of the Flanges***

Local plate bending in the flanges should be verified as



$$\frac{M_{Ed}}{M_{Rd}} + \left(\frac{N_{Ed}}{N_{Rd}}\right)^2 \le 1$$

$$M_{Ed} = m_{Ed} + w_{zEd}d \ \ and \ \ N_{Ed} = w_{yEd}$$

$$M_{Rd} = 0.2875\, t^2 \frac{f_y}{\gamma_{M0}} \text{ and } N_{Rd} = t.f_y/\gamma_{M0}$$

Outstand of flange d = **219.0** mm

Thickness of flange t = **37.0** mm

m_{Ed} = **46.0** KN.m

W_{yEd} = **20.0** KN

W_{zEd} = **50.0** KN

M_{Ed} = **57.0** KN.m

$N_{Ed} = W_{yEd}$ = **20.0** KN

$M_{Rd} = 0.2875\, t^2 \frac{f_y}{\gamma_{M0}}$ = **160.1** KN.m

$N_{Rd} = t \cdot \frac{f_y}{\gamma_{M0}}$ = **15540.0** KN

M_{Ed}/M_{Rd} = **0.356**

N_{Ed}/N_{Rd} = **0.0013**

$M_{Ed}/M_{Rd} + (N_{Ed}/N_{Rd})^2$ = **0.3556** **< 1.0 (O.K)**

C. Section Properties for secondary sheet piles

Design of Combi-wall is based on the assumption that soil pressure is transferred to the primary elements by virtue of arching action within the soil and water pressures are transferred to primary elements through secondary sheet piles. The verification of secondary sheet pile may be omitted if all the following conditions are met:

a) wall thickness of the secondary sheet piles ≥ 10mm;

b) pressure difference acting on the sheet piles: ≤ *40kN/m^2,* corresponding to 4 m differential water head;

c) maximum clearance between the primary elements is 1.5m for Z-piles.

To fulfil the above condition, the section property of secondary sheet pile is contemplated as follows;

i) For present design case, differential water pressure is limited to 2.0m water head.

ii) Selecting the width of secondary sheet pile element as 750mm and the formation of double-Z pile as 1500mm and keeping the minimum thickness of pile element as 10mm, further verification is not required.

Further, thickness of sheet pile shall be verified for driving stresses.

D. Design of Tie Rods

i) *Design Parameters*

Force per meter length in wailing = 610.00 KN/m

Spacing of Tie-rods = 2.127 M c/c

Length of Tie-Rod (L) = 40.00 m

Yield strength of Tie-Rod (fy) = 500.00 Mpa

Modulus of Elasticity of Rod (E) = 200000 Mpa

Rate of corrosion under buried condition = 0.035 mm/year

Design life of tie rod = 50 Years

ii) *Tensile Resistance of Tie-Rod*

Force in wailing Beam P = 610.0 KN/m

Spacing of Tie-Rod S = 2.127 M c/c

Tensile force in Tie-Rod T = P x S = 1297.47 KN

Increasing tensile resistance by 20%

Design Tensile force Ta = 1.2 x T = 1556.96 KN

iii) *Tensile Resistance of Shaft (Tie-Rod)*

Tensile Resistance of shaft; Ta = 0.6 x fy x As

Net area of shaft As = Ta / 0.6 x fy = 5189.80 mm^2

Net Dia. of Shaft $D = \sqrt{\frac{4.A_s}{\pi}}$ = 82.00 mm

iv) *Corrosion Allowance*

Rate of corrosion = 0.035 mm/year

Total corrosion loss at one side = 1.75 mm

Sacrificial thickness required 2 x 1.75mm = 3.50 mm

Gross Dia. of Tie-Rod = 76mm + 3.5mm = 85.50 mm

v) *Tensile Resistance of Thread*

Tensile Resistance of thread; Ta = Kt x 0.6 x fy x As

For non-bending condition;	Kt	=	0.90	
Area under thread;	At = Ta / Kt x 0.6 x fy	=	5767.00	mm^2
Dia. under thread	$D = \sqrt{\frac{4.A_t}{\pi}}$	=	86.00	mm

(Assuming no corrosion loss in threaded area)

Use upset bars of 95 mm nominal Dia.

vi) *Elongation in Tie Rod*

Tensile force	Ta	=	1557.00	KN
Length of tie Rod	(L)	=	40.00	m
Net Dia. of Tie-Rod		=	92.00	mm
Net area of shaft	(A)	=	6575.50	mm^2
Elongation in rod	δ = (Ta x L)/ (A x E)	=	47.36	mm

vii) Rotation in wall due to elongation of Tie-Rods

Height of Wall upto Anchorage level	(H)	=	22.00	m
Considering the anchor wall as non-yielding element, Deflection in wall	(Y)	=	47.36	mm
	Y / H	=	0.0022	
		=	0.215%	

The rotation in wall satisfies the criteria of Active Earth Pressures.

E. Design of Anchor Wall

For anchor wall, Z- type sheet piles are considered. Applying the tie-rod pull on anchor wall (with a factor), the ultimate design loads reflected by the anchor wall are indicated below.

i) Design Parameters (for anchor wall)

Design compressive force (N_{ED}) (additional load adopted for design)	=	**100.0**	KN/m
Design bending moment (M_{Ed})	=	**620.00**	KN.m/m
Design shear force (V_{Ed})	=	**425.0**	KN/m
Pile Section	=	**Z- type**	
Pile yield strength (f_y)	=	**420**	N/mm^2
Pile Young's Modulus (Es)	=	**210000**	N/mm^2

Design life = **50.00** years

Sheet piling corrosion section loss :

All faces of sheet piling = 0.075 mm

Pile section properties :			**Full properties**		**Reduced properties** (allowing for corrosion)	
Pile type		=	**Z**		Z	
Pile section width	B	=	**1400**	mm	**1400**	mm
Length of flange	b	=	**361**	mm	**361**	mm
Flange thickness	t_f	=	**12.2**	mm	**12.05**	mm
Web thickness	t_w	=	**12.2**	mm	**12.05**	mm
Overall height of section	h	=	**460**	mm	**460**	mm
Web angle	α	=	**55.2**	degrees	**55.2**	degrees
Moment of Inertia	I	=	**59720**	cm^4/m	**55000**	cm^4/m
Elastic section modulus	W_{el}	=	**2600**	cm^3/m	**2400**	cm^3/m
Cross section	A	=	**187.2**	cm^2/m	**165**	cm^2/m
	EI	=	**125412**	$KN.m^2/m$	**115500**	$KN.m^2/m$
	β_B	=	1.0			
	β_D	=	1.0			

Shear Capacity

Project shear area (A_v) = $t_w(h - t_f)$ = **38.6** cm^2/m

$V_{plRd} = A_v f_y / \sqrt{3}\, \gamma_{M0}$ = **934.9** KN / m

Design shear force (V_{Ed}) = **425.0** KN / m ($V_{Ed} < V_{pl\,Rd}$)

Shear Capacity Satisfactory

$V_{Ed} / V_{pl\,Rd}$ = **0.45** **<** 0.5

Web Shear buckling resistance

$c = (h - t_f) / \sin\alpha$ c = **545.5** mm

$\varepsilon = \sqrt{235/f_y}$ ε = **0.75**

c / t_w = **45.27**

72ε = **53.86**

for, $c / t_w < 72\,\varepsilon$ **Web Shear buckling need not verified**

$\bar{\lambda} = 0.346 \frac{c}{t_w} \sqrt{\frac{f_y}{E}}$ = **N/A**

f_{bv} = **N/A** N/mm^2

$V_{b,Rd} = \frac{(h - t_f)\, t_w f_{bv}}{\gamma_{M0}}$ = **N/A** KN / m

Cross-section class

$e = \sqrt{(235/f_y)}$ ε = **0.75**

$\frac{b/t_f}{\varepsilon}$ = **40.05**

Cross-section class = **1 & 2**

However, structure is designed, considering Class 3 or elastic section.

Moment Capacity

Section modulus (W) = **2400.0** cm^3/m

$M_{cRd} = \beta_B W_{pl}\, {f_y}/{\gamma_{M0}}$ or $\beta_B W_{el}\, {f_y}/{\gamma_{M0}}$ = **1008.0** KN.m / m

$V_{Ed} / V_{pl\,Rd}$ = **0.45** < 0.50 **No Moment capacity reduction required;**

$\rho = \left(2.V_{Ed}/V_{plRd} - 1\right)^2$ ρ = **N/A**

β_B = **1.0**

$M_{v\,Rd} = \left[\beta_B W_{pl} - \frac{\rho A_v^2}{4t_w \sin\alpha}\right] \frac{f_y}{\gamma_{M0}}$ = **1008.0** KN.m/ m

Design moment Resistance (M_{CRd}) = **1008.0** KN.m /m

Design bending moment (M_{Ed}) = **620.0** KN.m /m **Moment capacity satisfactory**

__Combined Bending and Compressio__

Unsupported length (L) = **1.0** m

Effective buckling length factor = **1.0**

Effective buckling length (l) = **1.0** m

$N_{cr} = E\,I\,\beta_D\,\pi^2/\,l^2$ N_{cr} = **1237767** KN / m

N_{Ed}/N_{cr} < 0.04 N_{ED}/N_{cr} = **0.0001** **Buckling resistance need not to be verified.**

Buckling resistance check **N/A**

__Plastic design resistance__

$N_{pl\,Rd}$ = A f_y / γ_{M0} $N_{pl\,Rd}$ = **7862.4** KN / m

$N_{Ed}/N_{pl\,Rd}$ < 1.0 $N_{Ed}/N_{pl\,Rd}$ = **0.0127** **Axial capacity satisfactory**

Effect of axial force on plastic moment of resistance

Cross section class = **1 & 2**

Pile profile = **Z**

Axial load to plastic capacity ratio limit = **0.1** **No reduction in moment capacity required**

Reduced bending capacity due to axial load

Moment reduction factor (F_N) = **1.0** Based on cross-section class and pile profile above

Reduced design moment (M_{CRd}) = **620** KN.m / m

$N_{Ed}/N_{pl\,Rd}$ = **0.01**

$M_{N\,Rd} = F_N M_{CRd}\left(1 - N_{ED}/N_{pl\,Rd}\right)$ = **1008** KN.m / m

Design bending moment (M_{Ed}) = **620.00** KN.m / m

Combined axial, bending and shear force effects

V_{ED} = **425.0** KN / m

$V_{pl\,Rd}$ = **934.93** KN / m

V_{Ed} /$V_{pl\,Rd}$ = **45.46** %

$\rho = \left(2.V_{Ed}/V_{plRd} - 1\right)^2$ ρ = **N/A**

$f_{yRd} = (1 - \rho)\, f_y$ = **420.00** N/mm^2

$M_{CRd}\dfrac{f_{yRd}}{f_y}$ = **1008.00** KN.m / m

Design bending moment (M_{Ed}) = **620.00** KN.m / m

Moment capacity satisfactory

F. Verification for Geotechnical Stability of Anchor Wall

i) Design Parameters

Force at anchor level (Tu) = 610.0 KN/m

Finished ground level = + 6.0 m Lvl.

Top Level of anchor wall = + 5.0 m Lvl.

Bottom Level of anchor wall = - 5.0 m Lvl.

Total depth of anchor Wall (D) = 10.0 m

Level of Tie rods for anchorage = + 2.5 m Lvl.

Depth of Fixidity of Wall (Do) = - 3.0 m Lvl.

GWT (adopted for design) (hw) = + 2.0 m Lvl.

Design Live Load Surcharge on backfill (q) = 50.0 KN/m^2

Spacing between Tie-Rods = 2.5 m c/c

ϕ of Backfill & Founding material = 32 deg.

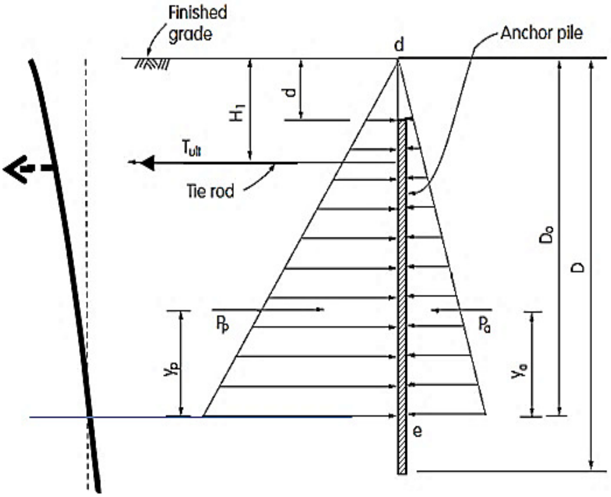
Unit wt. of soil (γ') = 18.0 KN/m^3

K_0 = 0.47

Ka = 0.31

Ka = 3.25

ii) Calculation of Forces


Finished grade
Anchor pile
d
d
H1
T_ult
Tie rod
Do
D
Pp
Pa
yp
ya
e

From above figure

Tu = 610.0 KN

H1 = 3.5 m

D = 11.0 m

Do = 9.0 m

d = 2.0 m

since this is a continous interlocked wall, the effective width of wall (b) is considered as the spacing of Tie-rods

D < b; force will be applicable upto the finished G.L.

$\sigma_a(1) = Ka.\gamma.Do$ = 49.78 KN/m^2

Normal Active Earth Pr. P1$= 0.5.K_a.\gamma.Do^2$ = 223.99 KN /m

C.G of active earth Pr. From point 'e' (y_a) = 3.00 m

Vertical Pr. due to Live Load surcharge = Ka .q = 15.36 KN/m^2

C.G of pressure due to surcharge from point 'e' (y_s) = 4.50 m

Active Earth Pr. due to Live Load surcharge Hs = Ka .q. D = 138.27 KN/m

Total active eath Pr. = Pa = P1 + Hs = 362.3 KN/m

Normal Passive Earth Pr. $Pp = 0.5.K_p.\gamma.Do^2$ = 2372.6 KN/m

C.G of passive earth Pr. From point 'e' (y_p) = 3.00 m

iii) Check for Stability of Wall

Natural stablising force on anchor wall Pp = 2372.6 KN/m

Total destablising force on Wall = Pa + Hs + Tu = 972.3 KN/m

$$Pp > Pa + Hs + Tu$$

F.O.S. in wall against horizontal forces = 2.44 (O.K)

$$T_{ult} = \frac{(P_p.y_p - P_a.y_a)}{(Do - H_1.)}$$

= 7117.8 KN.m

= 671.97 KN.m

= 622.2 KN.m

= 1058.8 KN/m

F.O.S. in wall against moment due to forces = 1.74 (O.K)

G. Stability Analysis of overall Anchored wall by Kranz Method

Ht. of wall above mud line (H) = 24.0 m

Depth of wall below mud line (D) = 14.0 m

Depth of anchor wall (h) = 12.0 m

Width of Bulkhead (L) = 40.0 m

Live Load Surcharge on bulkhead = 25.0 KN/m^2

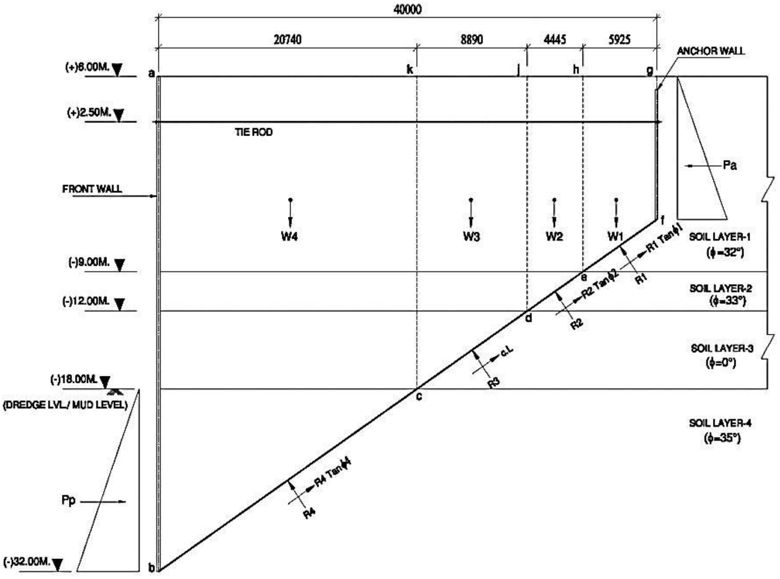

40000
20740
8890
4445
5925
ANCHOR WALL
(+)6.00M.
a
k
j
h
g
(+)2.50M.
TIE ROD
Pa
FRONT WALL
W4
W3
W2
W1
f
R1 Tanϕ1
SOIL LAYER-1 (ϕ=32°)
R1
(-)9.00M.
e
R2 Tanϕ2
SOIL LAYER-2 (ϕ=33°)
(-)12.00M.
d
R2
SOIL LAYER-3 (ϕ=0°)
c.L
R3
(-)18.00M.
(DREDGE LVL./ MUD LEVEL)
c
SOIL LAYER-4 (ϕ=35°)
R4 Tanϕ4
R4
Pp
(-)32.00M.
b

Figure E-9.6 : Force Diagram by Kranz Method

i) Calculation of vertical pressure s along the slip plane

- Average pressure due to soil wedge a-b-c-k;
  (4.0m x 17 KN/m^3 + 11.0m x 7 KN/m^3 +3.0m x 8 KN/m^3+ 6.0m x 11.0 KN/m^3 + $14.0\text{m}/2$ x 10.0 KN/m^3 = **305 KN/m^2**

- Average pressure due to soil wedge c-d-j-k;
  (4.0m x 17 KN/m^3 + 11.0m x 7 KN/m^3 + 3.0m x 8 KN/m^3 + $^{6.0\text{m}}/_{2}$ x 11.0 KN/m^3 = **202 KN/m^2**
- Average pressure due to soil wedge d-e-h-j;
  (4.0m x 17 KN/m^3 + 11.0m x 7 KN/m^3 + $^{3.0\text{m}}/_{2}$ x 8 KN/m^3
  = **157 KN/m^2**
- Average pressure due to soil wedge e-f-g-h;
  (4.0m x 17 KN/m^3 + $(7.0\text{m} + 11.0\text{m})2$ x 7 KN/m^3 = **131 KN/m^2**

Wt. of soil wedge (Wi)

W4 (soil wedge a-b-c-k)	20.74m x 305 KN/m^2	= 6325.7 KN
W3 (soil wedge c-d-j-k)	8.89 m x 202 KN/m^2	= 1795.8 KN
W2 (soil wedge d-e-h-j)	4.45 m x 157 KN/m^2	= 697.9 KN
W1 (soil wedge e-f-g-h)	5.92m x 131 KN/m^2	= 776.2 KN

Total wt. of soil wedge (W4 +W3+W2 + W1) **9595.5 KN**

Adding live load surcharge = 40.0m x 25 KN/m^2 = 1000.0 KN

Total vertical Load = 10595.5 KN

Normal Reaction of soil wedge along slip plane (W.cosθ)

θ = 34 deg;

W_4.cosθ = 5244.2 KN

W_3.cosθ = 1488.8 KN

W_2.cosθ = 578.6 KN

W_1.cosθ = 643.5 KN

Resistance due to each soil wedge along slip plane

W_4:	W_4.cosθ x tanφ_4 ($\varphi_4 = 35$ deg.)	= 3672 KN
W_3:	$c.L$ ($\varphi_3 = 0$ deg.)	= 1072 KN
W_2:	W_2.cosθ x tanφ_2 ($\varphi_2 = 33$ deg.)	= 376 KN
W_1:	W_1.cosθ x tanφ_1 ($\varphi_1 = 32$ deg.)	= 402 KN

Total = 5522 KN

From figure E-9.6 above;

Destabilising Force (F_D)

$= \Sigma W.\ \sin\theta + Pa + Ph$

$= \Sigma W.\ \sin\theta + {}^1/_2 Ka.\gamma.h^2 + Ph$

Resisting Force (F_R)

$= \Sigma W_i.\ \cos\theta\ .\ \tan\phi\iota + Pp$

$= \Sigma W_i.\ \cos\theta\ .\tan\phi\iota + {}^1/_2 Kp.\gamma.D^2$

$Ka = 0.30$ (for layer-1)

$Kp = 3.46$ (for layer-4)

$Pa = {}^1/_2\ Ka.\gamma.h^2 = 394.4$ KN/m length

$Pp = {}^1/_2 Kp.\gamma.D^2 = 6095.8$ KN/m length

Hydrostatic Pressure; $Ph = 520.0$ KN/m length

$\theta = 34.0$ deg.

$\sin\theta = 0.559$

$\cos\theta = 0.829$

$F_D = \Sigma W.\ \sin\theta + Pa + Ph = 6839.4$ KN

$F_R = \Sigma W_i.\ \cos\theta\ .\ \tan\phi\iota + Pp = 11618.8$ KN

F_R / F_D = (F.O.S against Sliding) **= 1.70 > 1.5 (O.K)**

H. Verification of stability of wall against various Modes of Failure

i) Deep-seated Failure

This type of failure is independent of the structural characteristics of the wall and/or anchor. The adequacy of the system (i.e. factor of safety) against this mode of failure is assessed by conventional slope stability analysis.

Applying Bishop's method for slope stability analysis, F.O.S is 3.26 > 1.5; Hence, it is safe.

ii) Rotational Failure (Toe Failure)

During a toe failure, the soil mass in front of the wall does not provide enough lateral resistance to resist the load applied due to the soil and surcharge loadings behind the wall. This type of failure is prevented by ensuring the adequate passive resistance by the embedded length of the wall.

From figure E-9.4;

Passive resistance in wall F_R = 6095.8 KN/m
[Refer E(ii)-anchor wall stability calculation]

From computer analysis results, total reaction at the base of wall in forward direction F_D = 1353.0 KN/m

F_R/F_D = 6095.8 / 1353.0 = 4.50 > 2.0; Hence, it is safe.

iii) Wale System Failure

Wale system failure is a separation of the anchor wall from the waling beam. This type of failure mechanism is a function of the oading conditions and connection detailing. **It should be taken care adequately during design and detailing of connections.**

iv) Anchor Failure

Anchor failure is the large movement of anchorage system. This type of failure mechanism is a function of soil strength and related passive resistance of wall. In order for an anchorage system to be effective it is located outside the potential active failure zone behind a sheet pile wall. The lateral pull capacity of anchor wall should be analysed by conventional means

Refer section F & G for Anchor stability analysis and it has adequate margin for F.O.S against instability; Hence, it is safe.

v) Tie rods Failure

Tie rod failure is the yielding and/or fracture of the steel tie rods connecting the sheet pile wall and the anchor wall placed at back. This type of failure mechanism is a function of the loading conditions, steel rod properties and deterioration. During tie rod failure, the anchor force required to support wall equilibrium exceeds the capacity of the tie rods.

For safety against tie rod failure following steps in design and detailing are considered.

i) Protection of tie-rods should be ensured by wrapping of tape or providing encapsulation of protective Tape with primer.

ii) Extra sacrificial thickness is provided considering loss of section due to corrosion in long term.

iii) The calculated tie tension is increased by 20 percent to account the load due to unforeseen geological factors etc.

iv) Suitable connectors, hinges and articulations/ fixtures should also be provided in tie rod connections to avoid bending stress and enhance the efficiency of joints.

It is safe against tie-rod failure.

vi) Flexural Failure of sheet pile wall

Flexural failure of wall occurs when the stress in the main structure of the wall is high enough to develop a plastic hinge. This type of failure mechanism is a function of the loading conditions, and structural design taking care of deterioration due to corrosion effect.

Geometrical section of wall is decided after taking care of all loads and corrosion loss; hence, it is safe against flexure failure.

Chapter 10

Load Bearing Capacity of Driven Piles

> *It is not the beauty of a structure one should look at; it's the construction of the foundation that will stand the test of time.*
>
> *David Allan Coe*

10.1 General

Steel piles are robust, light columnar elements, capable of carrying high compressive loads when driven on to a hard stratum, and capable of being driven to a deep penetration to reach a bearing stratum or to develop a high frictional resistance. They can be designed as a small displacement piles or large displacement piles.

Large displacement piles are generally a solid-section pile or close ended hollow-section piles (with pile cap), which are driven into the ground and thus displace the soil. All cast-in-place and driven types of piles with cap at bottom (as shown in **Figure 10-1**) come into this category.

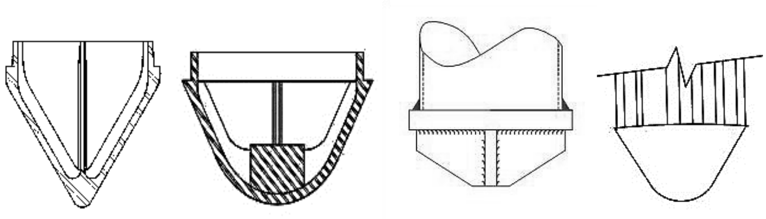


Figure 10.1 Pile Caps for Large Displacement Piles

Small displacement piles are also driven into the ground but have a relatively small cross-sectional area. They include rolled steel H- or I-sections and pipe or box sections driven with an open end such that the soil enters the hollow section. Where these types of piles get plugged with soil at bottom, they start behaving like large displacement piles.

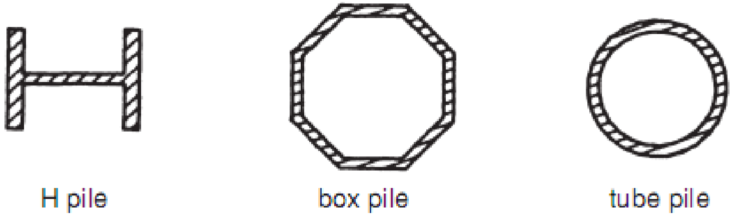


Figure 10.2 Open Ended Small Displacement Piles

Open ended steel pipe piles are used for various applications in costal engineering, port structures and increasingly important for offshore structures. They are highly preferred for deep offshore marine applications, where they can be fabricated and driven in large diameters to resist the lateral forces in deep water structures. Circular shape is also advantageous in minimising the drag and oscillations from waves and currents. Steel pipe piles consist of either rolled straight seam or spiral welded pipes. Spiral welded pipe piles (SSAW - Spiral Submerged Arc-Welded pipe piles) can be manufactured in various diameters with wall thicknesses up to 25 mm (1 inch). It is generally considered as most economical process for fabricating pipes. Rolled steel plate straight seam pipes (LSAW - Longitudinal Submerged Arc-Welded pipe piles) can be manufactured in various diameters and may have wall thicknesses up to 100mm or more.

10.2 Driven Pipe Piles for Foundations

In recent times, open-ended steel pipe piles have gained popularity, particularly in marine and offshore deep foundations due to smaller driving resistance compared to closed-ended piles. Driven steel piles can be effectively used to reduce project costs, shorten project schedules, and reduce environmental concerns.

10.3 Load Bearing Capacity of Pipe Pile Foundations

The basis of the ‘soil mechanics approach’ to calculate the load carrying capacity of driven piles is that the total resistance of the pile to compression load is sum of the two components, namely **shaft friction** and **base resistance**.

During pile driving, side resistance develops on both the exterior and interior surface of an open-end pile as the pile penetrates through the soil. As side resistance builds up on the pile interior, especially if compaction and arching of the soil occurs, then a soil plug may form within the pile. Once plugged, the pile behaves as a displacement pile and the base resistance acts over the entire end area. If a pile does not plug during driving, base resistance acts only on the pile annulus, and interior pile side resistance may be considered in the design.

Thus, the term soil plugging generally refers to the state where the soil within the pile has mobilized enough frictional force to overcome the toe resistance and prevent additional soil from entering the pile as illustrated in **Figure-10.3(a)**.

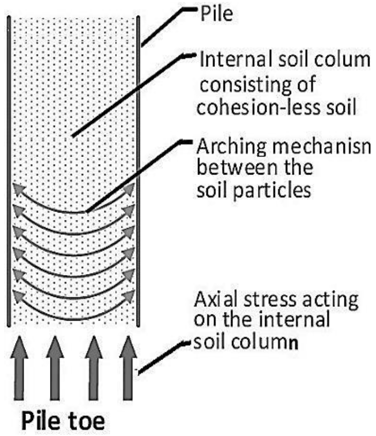

Pile
Internal soil colum consisting of cohesion-less soil
Arching mechanisn between the soil particles
Axial stress acting on the internal soil column
Pile toe

Figure 10.3 (a) Illustration of arching principle of soil in pipe pile.

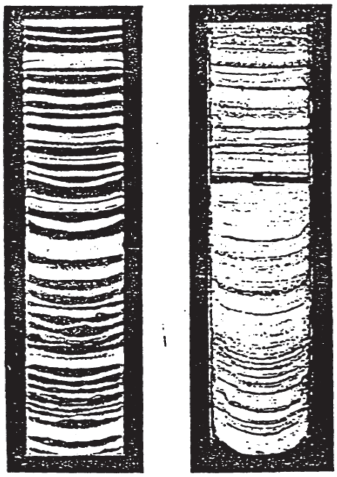


Figure 10.3 (b) Illustration of passive arching (Paikowsky, 1989).

Plug development has potentially different behaviour during pile driving and under static loading. During pile driving, soil on the pile interior may exhibit inertial resistance to downward pile acceleration, and thereby prevent formation of a pile plug. However, under static loading conditions when the pile is not rapidly moving, the soil may indeed act as a plug within the pile interior (Lehane and Randolph 2002).

Plug development in pipe pile has complex nature and requires careful consideration during pile design and installation. The mechanism of soil plug formation in different type of soil is described below.

10.3.1 Soil Plugging Mechanism in Cohesion-less Soil

Soil plugging is generally a state where the soil within the pile has mobilized enough frictional force to overcome the toe resistance of pile. The mechanism for redistribution of stresses within the soil body has been shown by Paikowsky (1989) and the arching mechanism of material within a confined space can be well explained by Janssen theory for Silo. During driving of open-ended piles in a laterally confined space, the pile walls move relative to the soil body, referred to by Paikowsky (1989) as passive arching (as opposed to active arching in the Silo theory). Passive arching in an open-ended pile will cause concave soil formation at the pile toe level, shown in **Figure 10-3(b)**. In this case the arching mechanism is able to transfer axial stress acting on the internal soil column at the toe of the pile to the pile walls in the form of horizontal (normal) stress, giving rise to increased internal shaft friction.

Figure 10-4 presents a schematic detail of fully-plugged, partially-plugged and unplugged pile conditions. If a pile does not plug during driving, base resistance acts only on the pile annulus, and interior pile side resistance may be considered in the design.

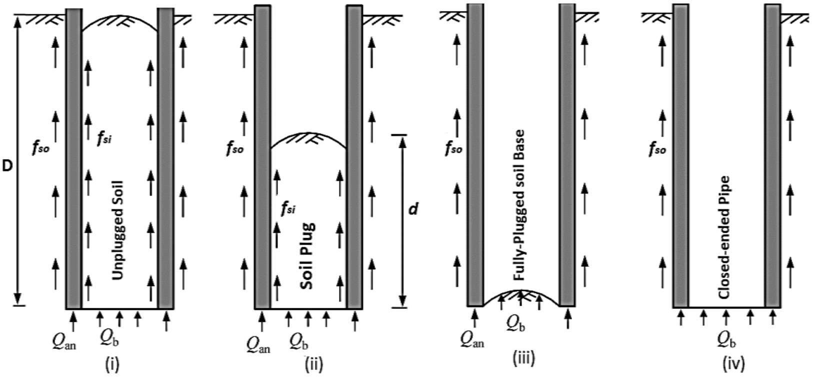


Figure 10.4 State of Penetration and plugging of Pipe Pile in soil

(i) Fully-unplugged pile base
(ii) Partially-plugged pile base
(iii) Fully-plugged pile base
(iv) Closed-ended Pipe

Formation of soil plug in a driven pile is governed by number factors and above cases of plugged, unplugged and partially plugged conditions of pile. Following pile installation related factors must be considered that affect the design and performance of open-ended tubular piles including soil plug formation.

i) **Relative density of soil** - Plugging of open-end piles generally occur in medium dense to dense sands and in stiff clay. In case of loose sand deposits of soft clay, the plugging effect is insignificant.

ii) **Ratio of pile penetration-to-dia. (L/D)** - Plugging of open-end piles begins when the pile penetration-to-diameter ratio reaches greater than 20 in medium dense to dense sand strata. Plugging may occur in stiff clay strata for pile penetration-to-diameter ratios of 10 to 20, but that the clay plug may not contribute significantly to the total pile resistance.

iii) **Driving methods -** Vibratory pile installation causes remoulding of cohesive soils near the pile wall or liquefaction of cohesionless soils at the pile/soil interface may hinders the plugging effect. Plugging is effective in impact driving. Generally, initial pile section is preferred to be installed using vibratory hammers and impact hammers are used for the final pile section to develop the plug.

iv) **Dia. of pile** - Higher degree of plugging happens in relatively smaller dia. piles. German Geotechnical Society distinguishes between three cases:

   a. fully plugging is possible for piles having Dia. $\leq$ 0.5m;

   b. no plugging effects are observed in piles with Dia. $\geq$ 1.5m; and

   c. Partial plugging occur in piles with dia. (D) (0.5m < D < 1.5m)

v) **Use of driving shoe** - A driving shoe, consisting of a ring of steel with larger thickness at the pile toe is provided to allow the pile penetration into a hard stratum. Driving shoes are typically installed within the pile interior, which results in a reduced interior pile diameter at the pile tip. This reduced interior diameter may subsequently result in reduced interior side friction and prohibit the development of a pile plug.

vi) **Set-up effect of soil**: It is a time dependent process. As a pile is driven into the ground, it disturbs the surrounding soils and when it recovers from the installation disturbance, a time-dependant increase in pile capacity often occur known as “setup”.
Setup usually occurs in cohesive soils (clay), or a mixture of fine-grained granular and cohesive soils (clayey sand, or clayey silt). in fine-grained granular soils (fine sand, or non-plastic silt), it is not very significant and insignificant for coarse grained soil.

10.3.2 Design Considerations for Pipe Piles

To evaluate the load bearing capacity of driven pile system, thorough information of the prevailing ground conditions, pile installation method and associated loading are important. This behaviour is influenced greatly by the method used to install the piles and it cannot be predicted solely from the physical dimensions of the pile and of the undisturbed soil properties. The estimation of their load-carrying capacity is based geo-mechanics approach using empirical correlations involving numerical analysis and field tests. It largely depends on the inner frictional contribution determined by soil plugging effects.

A simplified approach to estimate the bearing capacity of an open-ended pile consists of following parameters

Frictional resistance from outer shaft of pile (Q_{out})

End bearing from annulus of pile and (Q_{an})

Frictional resistance from inner surface of pile (Q_{in}) or bearing capacity of soil plug (Q_b)

$$Q_u = Q_{an} + Q_{out} + Q_{plug} \tag{10.1}$$

Where, Q_u is bearing capacity, Q_{an} is annulus resistance, Q_{out} is outer frictional resistance and Q_{plug} is plug resistance (see Eq. (10.2)).

$$Q_{plug} = \min\left(Q_{in}, Q_b\right) \tag{10.2}$$

Where, Q_b is base resistance.

10.3.3 Inner surface Frictional Resistance

During the driving process, open-end piles develop side resistance on both the pile interior and exterior. For piles that remain unplugged at the end of driving, the interior pile surface resistance may be accounted for in design. There are varying recommendations in the literature about how to evaluate the interior pile side resistance. Observations and guidelines on the evaluation of interior pile side resistance include:

- ***Design guidance from the American Petroleum Institute (API)*** generally recommends that the interior pile side resistance should be considered equal to that of the pile exterior (API 2011). However, API also recommends that the internal side resistance contribution should be limited to a maximum value equal to the base resistance acting on the soil plug according to Eqn-10.2
- ***The Norwegian Geotechnical Institute (NGI) design method*** recommends interior pile side resistance acting on the soil plug is considered equal to three times the external value due to soil arching near the pile base (Clausen et al. 2005).

- ***Randolph et al. (1991)*** discuss that values of internal side resistance have historically been assumed to be comparable to values of external side resistance. The internal side resistance under dynamic loading is similar to residual skin friction on the pile exterior. It is observed that the arching action within the pile results in very high values of internal skin friction.
- ***Lehane and Randolph (2002)*** discuss the dependence of internal side friction on the dilatant capacity of the sand plug and the interface friction angle. The authors provide a range of typical friction coefficient β values from 0.1 to 0.6.
- ***Lehane and Gavin (2001)*** recommend the interior pile minimum friction coefficient β be calculated as $\beta_{min} = K_a \cdot \tan\delta$, where K_a is the Rankine active earth pressure coefficient and δ is the interface friction angle.

10.4 Computation of Allowable Axial Resistance

10.4.1 Static Axial Resistance Design Methods in soil

Numerous static axial resistance design methods are available for calculation of predicted pile nominal resistance. These include well-known methods used for foundation design and infrastructure projects as provided below;

In recent years, new methods have been published specifically developed for open-end piles such as

i) Tomlinson -method
ii) API method (total stress approach)
iii) Gudavalli method

Table -10.1: Selected methods for surface (skin) resistance design of driven piles.

Method	**Design Equations**	**Equation Parameters**
Tomlinson α (Tomlinson 1980)	$f_s = \alpha . S_u$	α = adhesion factor S_u = undrained shear strength
API α (API 2011)	$f_s = \alpha . S_u$ α is defined as: $\alpha = 0.5 \cdot \Psi^{-0.5}$ for $\Psi \leq 1.0$ $\alpha = 0.5 \cdot \Psi^{-0.25}$ for $\Psi > 1.0$ where, $\Psi = Su / (\sigma' v)$ and $\alpha \leq 1.0$	α = adhesion factor S_u = undrained shear strength σ'_v = vertical effective stress
API β (API 2011)	$f_s = \beta . \sigma'_v$	β = friction coefficient σ'_v = vertical effective stress
Gudavalli (Gudavalli et al. 2013)	For dense sand to very dense sand: $f_s = \beta . \sigma'_v$ $\beta = (3.5 - 3.2\, PLR)^{-0.023\, l}$ $PLR = \left(\frac{bi}{1.4}\right)^{0.19}$	β = friction coefficient σ'_v = vertical effective stress PLR= plug length ratio ℓ= pile penetration depth (m) *bi* = interior pile diameter (m)

Table 10.2: Selected Base Resistance Design Methods.

Method	**Design Equations**	**Equation Parameters**
Total stress Approach (API 2011)	$q_b = N_c . S_u$ where, N_c= 9	S_u = undrained shear strength N_c = bearing capacity factor
Bearing capacity Approach (API 2011)	$q_b = N_q \cdot \sigma' \mathrm{v}$	N_q = bearing capacity factor σ'_v = vertical effective stress
(Gudavalli et al 2013)	For dense to very dense sand: $q_b = N_q \cdot \sigma' \mathrm{v}$ $N_q = 12.3\ PLR^{-8.4}$ $PLR = \left(\frac{bi}{1.4}\right)^{0.19}$	N_q = bearing capacity factor σ'v = vertical effective stress PLR = plug length ratio bi = interior pile diameter (m)

10.4.2 Analytical Approaches

(i) API method

The American Petroleum Institute (API) (2007) suggests a conservative quasi-empirical correlation to calculate the ultimate axial bearing capacity of open-ended pipe piles. It recommends that the interior pile side resistance should be considered equal to that of the pile exterior (API 2011). However, API also recommends that the internal side resistance contribution should be limited to a maximum value equal to the base resistance acting on the soil plug.

In the unplugged condition, base resistance acts over the pile annulus only, while base resistance will develop over the total cross-sectional area for plugged piles. It is common practice, as recommended in the API guidance and other sources, to compute base resistance as the minimum value of the plugged resistance acting over the total cross section and the sum of the unplugged annular resistance and the internal pile side resistance (API

2011 and Clausen et al. 2005). The values used to calculate the capacity of the pile are based on experience for a conservative design.

To determine the axial bearing capacity of open steel pipe piles the API suggests the following formula

$$Q_u = Q_b + Q_f = q_b A_b + f_s . A_{sj} \tag{10.3}$$

Where,

Q_u = Ultimate axial bearing capacity,

Q_b = end bearing,

Q_f = Skin frictional resistance or shaft resistance,

q_b = unit end bearing capacity,

$A_b = \pi D^2 /4$,

f_s= unit shaft friction,

A_s = corresponding side surface area of the pile and

j = the different soil layers.

In Cohesive soils, the unit end bearing Q_b, in kPa, may be computed by the equation

$$q_b = N_c . S_u$$

Su = undrained shear strength

Nc = bearing capacity factor which is equal to 9.

For pipe piles in cohesive soils, the shaft friction, *f* (kPa) at any point along the pile may be calculated by the equation.

$$f_s = \alpha . S_u$$

α is an adhesion factor defined as:

$\alpha = 0.5 \cdot \Psi^{-0.5}$ for $\Psi \leq 1.0$

$\alpha = 0.5 \cdot \Psi^{-0.25}$ for $\Psi > 1.0$

Where, $\Psi = Su / (\sigma' v)$ and $\alpha \leq 1.0$,

Su = undrained shear strength

$\sigma' v$ = vertical effective stress

For piles driven in cohesionless soils, the API proposes the unit end bearing q_b (kPa) may be computed by the equation,

$$q_b = N_q \cdot \sigma' \mathrm{v}$$

N_q = bearing capacity factor

$\sigma' \mathrm{v}$ = vertical effective stress

As per API, the recommended values of *Nq* are presented in Table-10.3.

For pipe piles in cohesionless soils, the shaft friction, *f*, and (kPa) may be calculated by the equation:

$$f_s = \beta . \sigma'_v$$

σ'_v = vertical effective stress

β = friction coefficient

β = $Kp_o \tan \delta$

Where, *K* = coefficient of lateral earth pressure (ratio of horizontal to vertical normal effective stress). For open-ended pipe piles driven unplugged, *K*= 0.8 (for both tension and compression)

For plugged or closed end piles *K* = 1.0.

Po = effective overburden pressure,

δ = friction angle between the soil and pile wall. The value of δ may be referred from Table 10.3; if other data are not available.

The above formula for the unit shaft friction at a given depth z and the unit end bearing capacity;

$$f = \beta \sigma v_0 (z) < \tau_{max} \quad (10.4)$$

$$q_b = Nq. \sigma v_0 ' (z) < q_{max} \quad (10.5)$$

$$\sigma'_{vo} = \gamma_{sub} . z$$

γ_{sub} = submerged unit weight of the soil,

τ_{max} = maximum unit shaft friction,

q_{max} = maximum unit end bearing.

The values of τ and q are limited due to the fact that the shaft friction and end bearing for long piles do not increase linearly with the overburden pressure.

Table - 10.3: Design Parameters for Cohesionless Soil

Density	**Soil Description**	**Corrected SPT Value ' N_{60} ' ****	**Soil-Pile Friction Angle, δ (Degrees)**	**Limiting Skin Friction Values τ_{max} (kPa)**	N_q	**Limiting Unit End Bearing Values q_{max} (MPa)**
Very Loose Loose Medium	Sand Silty Sand / Sandy Silt Silt	4 < 4 - 10 10 - 20	15	47.8	8	1.9
Loose Medium Dense	Sand Silty Sand / Sandy Silt Silt	4 - 10 10 - 30 20 - 40	20	67	12	2.9
Medium Dense	Sand Silty Sand / Sandy Silt	10 - 30 30 - 50	25	81.3	20	4.8
Dense Very Dense	Sand Silty Sand / Sandy Silt	30 - 50 > 50	30	95.7	40	9.6
Dense Very Dense	Gravel Sand	≥ 40 > 50	35	114.8	50	12.0

*** Values are empirically co-related*

Modified API

The degree to which plugging has occurred is, as mentioned previously, often described by the incremental filling ratio, IFR. Another, somewhat more basic, method of capturing the plugging behaviour is the plug length ratio, PLR. Both methods are related and as is illustrated in **Figure 10-5** the IFR value is the first derivative of PLR. A large study was presented by Gudavalli, et al. (2013), specifically aimed at analysing open-ended pipe piles driven by impact hammer into dense and very dense sands. The purpose of the study was to correlate the pile diameter and depth of penetration to values of PLR, β (used to calculate shaft resistance) and a bearing capacity factor, Nq to calculate toe resistance.

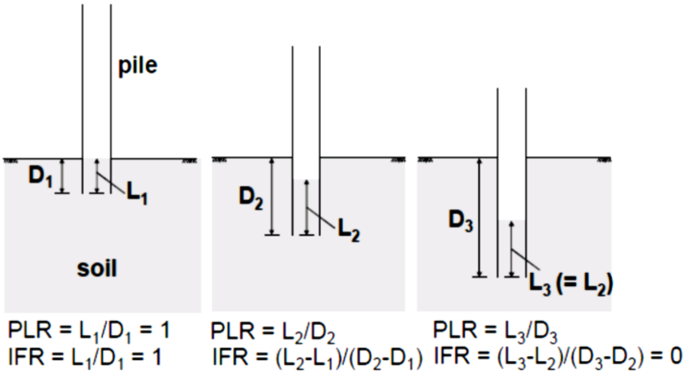


Figure 10.5 Definition of Plug Length Ratio (PLR) and Incremental Filling Ratio (IFR) used to describe soil plugging (Gudavalli, et al., 2013).

The method by Gudavalli et al. (2013) suggests improved design values for the API. By analysing 1355 test piles (406mm < d < 914mm) Gudavalli et al. obtained an improved equation for the end bearing factor which is valid for driven piles installed into dense to very dense sand with a range of PLR from 0.76 to 0.91.

For internal pile diameters between 0.387 m and 0.876 m, the PLR can be estimated from:

$$PLR = \left({}^{d}/_{1.4}\right)^{0.19} \tag{10.6}$$

A similar relationship was presented by Lehane, et al. (2005) where the average value of IFR (corresponding to PLR) over a length of 20 pile diameters could be estimated according to:

$$PLR = \left({}^{d}/_{1.4}\right)^{0.2} \tag{10.7}$$

$$N_q = 12.3\,PLR^{-8.4} \tag{10.8}$$

The results from the study presented by Gudavalli, et al. (2013) show a clear relationship between pile diameter and PLR value, where a smaller pile diameter corresponds to a lower PLR and vice versa. A lower PLR value indicates that a higher degree of plugging has taken place, this is in agreement with the general consensus that long and slender piles are more prone to plugging compared to wider and shorter piles. The mean values of PLR in this study ranged from 0.76 - 0.91.

The value of β can be estimated from:

Gudavalli et al. suggests the following formula to estimate the value of β

$$\beta = (3.5 - 3.2\,PLR)^{-0.023\,l} \tag{10.9}$$

If no PLR could be measured during the pile driving process, it can be estimated as follows

$$PLR = \left({}^{d}/_{1.4}\right)^{0.19} \tag{10.10}$$

Where, 0.378 m < d < 0.876 m.

(ii) Harmonization of EA Piling and EAU

Lüking & Becker (2015) propose a method for the axial bearing capacity based on the calculation methods for open steel pipe piles of EA-Pfähle (2012) and EAU (2012). Both, EA- fähle, Recommendation on Piling, and EAU, Recommendations on waterfront structures, harbours and waterways, are elaborated by the German Geotechnical Society and widely-used in Germany and partly Europe.

The method distinguishes between three cases: fully plugged ($D \leq 0.5$m), no plugging effect ($D \geq 1.5$m) and partially plugged (0.5m < D < 1.5m).

The total axial bearing capacity for a fully plugged open steel pipe pile reads

$$Q_{u1} = \eta b. q_b. A_p + q_b. A_{an} + \eta_s. f_s. A_s \qquad (10.11)$$

Where, ηb = $2.52e^{-1.85D}$; with D in [m],

q_b = bearing resistance of the plug [kPa]
A_p = value of the area of the plug [m^2],
q_b = bearing resistance of the annulus [kPa] (cf. Fig. 1),
A_{an} = area of the annulus [m^2],
η_s = $1.53\ e^{-1.85D}$; with D in [m],
f_s = value of the shaft friction on the outside, of the pile
A_s = area of the outer shaft [m^2]

In the case of no plugging effect, the proposed formula reads

$$Q_{u2} = q_b. A_{an} + f_{si}. A_{si} + f_{sj}. A_{sj} \qquad (10.12)$$

Where,

f_{si} = characteristic value of the shaft friction on the inside for the layer of soil i [kPa]

A_{si} = area of the inner shaft area minus the top 0.2 L of the pile [m^2],

For a partially plugged pile both previous methods are combined

$$\text{Qu} = \Psi Q_{u1} + \chi\ Q_{u2} \qquad (10.13)$$

Where, Ψ and χ calculation factors considering the pile diameter D according to Fig. 3.a (b)

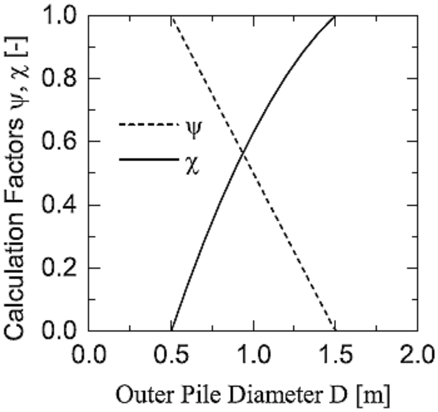


Figure 10.5 Calculation factors Ψ and χ according to Lüking & Becker (2015).

10.4.3 Steel H-Piles

Steel H-piles consist of rolled wide flange sections. In most H-pile sections, the flange and web thicknesses are the same. Steel H-piles are non-displacement piles and often used where a large number of piles are needed in a small area. Splicing of H-piles is also very easy job. Steel H-Piles are very effective when driven into soft rock. They can be driven very hard with modern high impact velocity hammers with little likelihood of pile toe damage.

In addition, the problems associated with soil heave during foundation installation are often reduced by using H-piles. However, sometimes H-piles will "plug". That is, the soil being penetrated will adhere to the web and the inside flange surfaces creating a closed-end, solid section. The pile will then drive as if it were a displacement pile below the depth of plug formation. Plugging can have a substantial effect on both the soil resistance during driving and the ultimate static pile capacity.

An H-pile in a granular profile will often not plug during the dynamic loading of pile installation but may plug under the slower static loading condition. The plugging phenomenon in H-piles can be equally difficult to analyse. However, the distance between flanges of an H-pile is smaller than the inside diameter of most open-end pipe piles. Therefore, it can usually be assumed that an H-pile will be plugged under static loading.

$$Q_u = 0.8\,\alpha\,A_s + 9.\,C_u A_b/2 \qquad (10.14)$$

Where:

Q_u = Ultimate pile capacity in kN

α= Pile adhesion from Table-10.1

A_s = Pile-soil surface area in m^2

C_u = Average undrained shear strength at the pile toe in kPa

A_b = Gross toe area of a plugged pile in $[m^2]$,

For H-piles, effective plugged area is around 50% of the gross area. Therefore, in H-piles, theoretical end bearing resistance should be calculated corresponds to $A_b/2$ as denoted in equation 10.14.

The toe capacity for H-piles driven to rock is usually governed by the pile structural strength, calculated based on the steel cross sectional area, and should not include the area of a soil plug, if any. For H-piles in cohesionless soils, arching between the flanges can usually be assumed, and the "box" perimeter can be used for shaft resistance calculations.

This is also the case if a plug forms inside the cross section of the sheet pile wall. However, there is not yet established a practice for estimation of the transferral of a downwards directed force to the soil if the steel area of the sheet pile wall cannot provide sufficient bearing capacity.

In Germany, the area is calculated according to the section angle α, refer **Figure 10.6**. The reduction factor κF is calculated according to Eq. 10.15 (Weissenbach, 1977, 2008) where α should be entered in degrees.

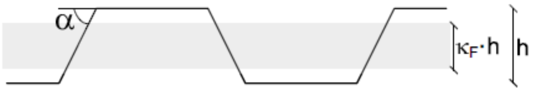


Figure 10.6 Calculation of effective height of a sheet pile wall according to Weissenbach, (1977, 2008)

$$K_F = 0.015 \cdot \alpha - 0.35 \qquad (10.15)$$

Sheet pile wall with U or Z type sheets are generally constructed with α between 39.5º and 81.5^{o}. This provides an effective area of 40% to 87% of the area of the circumscribed convex polygon. Before the theoretical evaluation of plugging, the magnitude of the forces at the pile toe should be investigated for sheet pile walls.

The plugging phenomenon is investigated by Paikowsky and Whitman (1990) for open-ended pipe piles and they found that a full plug occurs for a penetration to diameter ratio of 10−20.

EXAMPLE : 10.1

An open ended tubular pile (OD = 600mm with 18mm cell thickness) is driven through medium dense sand strata (φ = 33.0 deg. and bulk density of soil; γ = 18 KN/m^3). Compute the allowable load bearing capacity on pile by applying a factor of safety as 2.5 ultimate pile load capacity.
Consider the elevation of ground as +3.0m and G.W.T. at +1.0m level.

Solution:

The S.B.C of pile is based on API method.

Critical depth of pile for soil arching =20 x Dia of pile = **-9.0** M Lvl;

Length of pile (L) = **12.0** M

Outer surface area of pile /m run = **1.94** m^2/m run

Area of pile annulus (Ap) = **0.017** m^2

a) Skin friction of pile for (segment - 1)
(From EL. 3.0M to 1.0M)

Undrained shear strength; Su = **0.0** KN/m^2

Effective unit weight of soil (γ') = **18.0** KN/m^3

Vertical stress at top of strata (p_{r1}) = **0.0** KN/m^2

Vertical stress at bottom (p_{r2}) = **36.0** KN/m^2

Mean vertical effective stress σ_v = **18.0** KN/m^2

K = **1.0**

δ = **25** deg.

β = K.tanδ β = **0.47**

$f_s = \beta . \sigma_v$ f_s = **8.39** KN/m^2

Length of pile segment (L_1) = **2.00** m

Outer surface area of pile = **3.88** m^2

Skin friction for segment-1 $f_s . A_{so}$ = **32.59** KN

b) Skin friction of pile for (segment - 2)

EL. 1.0M to -9.0M

Undrained shear strength; Su = **0.0** KN/m^2

Effective unit weight of soil (γ') = **8.0** KN/m^3

Vertical stress at top of strata (p_{r1}) = **36.0** KN/m^2

Vertical stress at bottom (p_{r2}) = **116.0** KN/m^2

Mean vertical effective stress σ_v = **76.0** KN/m^2

K = **1.0**

δ = **25** deg.

β = K.tanδ β = **0.47**

$f_s = \beta . \sigma_v$ f_s = **29.36** KN/m^2

Length of pile segment (L_2) = **10.00** m

Outer surface area of pile $A_{so} = (\pi \text{ x } D_o \text{ x } L_2)$ = **19.42** m^2

Skin friction on outer surface of pile $f_s A_{so}$ = **688.06** KN

Similarly,

Skin friction on inner surface of pile $f_s A_{si}$ = **668.02** KN

$\Sigma f_s A_{so}$ = **720.65** KN

$\Sigma f_s A_{si}$ = **699.66** KN

Estimation of End Bearing Capacity of Pile

Considering the plugging effect in pile

Unit end bearing capacity at pile base $q_b = \text{Nq} . \sigma_v$

Nq = **30.0**

σ_v = **116.0** KN/m^2

q_b = **983.95** KN/m^2

= **0.983** MPa

< **4.80** Mpa; q_{max}

(O.K) Area of annulus at pile base

$A_{an} = \pi/4 \text{ x } (Do^2 - Di^2)$ = **0.0172** m^2

Area of soil plug at pile base

$A_b = \pi/4 \text{ x } Di^2$ = **0.283** m^2

Ultimate bearing capacity of at pile annulus

$Q_{an} = q_b \text{ x } A_{an}$ = **16.92** KN

Considering the plugging effect in pile

$Q_p = q_b \text{ x } A_b$ = **278.45** KN

Permissible soil plug capacity Q_p = **nin (Q_p , Σ f_s . A_{si})**

Q_p = **278.45** KN

Total end bearing capacity of pile = **Q_p + Q_{an}**

278.45 + 16.92

= KN

= **295.37** KN

Ultimate load capacity of pile Qu

$Qp \;+\; \boldsymbol{\Sigma f_s . A_{so}}$ = **1016.02** KN

Adopting F.O.S for S.B.C of Pile = **2.50**

Safe Bearing Capacity of pile = **406.40** KN

EXAMPLE: 10.2

Redesign the pile given in Example -10.1(using API method), if the pile is driven through the uniform clayey strata. The undrained shear strength of clay is 85 KN/m^2 and bulk density as 18 KN/m^3.

Solution:

Considering the critical depth in pile for arching of soil

= 20 x Dia of pile = **12.0** M

= **-9.0** M Lvl

Length of pile (L) = **12.0** M

Outer surface area of pile /m run = **1.94** m^2/m run

a) Skin friction of pile for (segment - 1)

From **EL. 3.0 M to 1.0 M**

Undrained shear strength; Su = **85.0** KN/m^2

Effective unit weight of soil (γ') = **18.0** KN/m^3

Vertical stress at top of strata (p_{r1}) = **0.0** KN/m^2

Vertical stress at bottom (p_{r2}) = **36.0** KN/m^2

Mean vertical effective stress σ_v = **18.0** KN/m^2

Referring equation in table 10.1;

$\Psi = S_u/\sigma_v$ Ψ = **4.7**

α = **0.34**

$f_s = \alpha . S_u$ = **28.83** KN/m^2

Length of pile in segment-1 (L_1) = **2.00** m

Outer surface area of pile A_{so} =(π x D_o x L_1) = **3.88** m^2

Skin friction on outer surface of pile($f_s . A_{so}$) = **24.03** KN

Length of pile segment (L_1) = **2.00** m

Outer surface area of pile (As x L_1) = **3.88** m^2

Skin friction for segment-1 $f_s . A_{so}$ = **111.95** KN

b) Skin friction of pile from (segment - 2)

EL. 1.0M to -9.0M

Undrained shear strength; Su1 = **85.0** KN/m^2

Effective unit weight of soil (γ') = **8.0** KN/m^3

Vertical stress at top of strata (p_{r1}) = **36.0** KN/m^2

Vertical stress at bottom (p_{r2}) = **116.0** KN/m^2

Mean vertical effective stress σ_v = **76.0** KN/m^2

Ψ = **1.1**

α = **0.49**

$f_s = \alpha . S_u$ = **69.50** KN/m^2

Length of pile segment (L_2) = **10.00** m

Outer surface area of pile A_{so} =(π x D_o x L_2) = **19.42** m^2

Skin friction on outer surface of pile

($f_s . A_{so}$) = **802.37** KN

$\boldsymbol{\Sigma f_s . A_{so}}$ = **914.32** KN

$\boldsymbol{\Sigma f_s = A_{si}}$ **887.69** KN

Estimation of End Bearing Capacity of Pile

Considering the plugging effect in pile

Unit end bearing capacity at pile base q_b = c . Nc

c = **85.0** KN/m^2

Nc = **9.0**

q_b = 85.0 x 9.0 = **983.95** KN/m^2

Area of annulus at pile base

$A_{an} = \pi/4 \times (Do^2 - Di^2)$ = **0.0172** m^2

Area of soil plug at pile base

$A_b = \pi/4 \times Di^2$ = **0.283** m^2

Ultimate bearing capacity of at pile annulus

$Q_{an} = q_b \times A_{an}$ = **13.17** KN

Considering the plugging effect in pile

$Q_p = q_b \times A_b$ = **216.30** KN

Permissible soil plug capacity Q_p = **min (Q_p , $\Sigma f_s . A_{si}$)**

Q_p **216.30** KN

Total end bearing capacity of pile = **$Q_p + Q_{an}$**

216.30 + 13.17 KN

= **229.47** KN

Ultimate load capacity of pile Qu

$Qp + \boldsymbol{\Sigma f_s . A_{so}}$ = **1143.79** KN

Adopting F.O.S for S.B.C of Pile = **2.50**

Safe Bearing Capacity of pile = **457.52** KN

EXAMPLE : 10.3

An open ended tubular pile (OD = 916 mm with 16mm cell thickness) is driven through stratified soil. The mechanical properties of soil and pile installation details are given in table below. Compute the allowable load bearing capacity on pile by applying API method. The applicable F.O.S for safe bearing capacity of pile is 2.5.

Properties of Sub-soil strata					
E.L. of Soil Strata	*Type of Soil*	*Depth (m)*	*SPT 'N'$_{60}$*	γ' *(KN/m^3)*	*c-value (KN/m^2)*
From E.L to E.L					

-6.0 m	*to*	*-10.0 m*	*Medium Sand*	*4.0*	*21*	*8.00*	*0.0*
-10.0 m	*to*	*-16.5 m*	*Medium Sand*	*6.5*	*25*	*9.00*	*0.0*
-16.5 m	*to*	*-18.0 m*	*Silty Clay of Low plasticity*	*1.5*	*7*	*9.00*	*40.0*
-18.0 m	*to*	*-28.0 m*	*Medium Sand*	*10.0*	*27*	*9.60*	*0.0*
-28.0 m	*to*	*-36.0 m*	*Dense Sand*	*8.0*	*40*	*10.00*	*0.0*

Installation Details of Pile

Top elevation of Ground		=	***- 7.0***	*M*
Founding level of Pile		=	***-34.0***	*M*
Ground water table		=	***(+) 1.0***	*M*
Unit weight of water		=	***10.00***	*KN/m^3*
Outer dia of pile	*(Do)*	=	***0.916***	*M*
Inner dia of pile	*(Di)*	=	***0.900***	*M*
Area of annular base	*(An)*	=	***0.023***	*m^2*
Area of plugged base for pile	*(Ab)*	=	***0.636***	*m^2*

Solution:

Referring table 10.1, 10.2 & 10.3; the limiting properties of soil for skin friction and end bearing are tabulated below.

E.L. of Soil Strata			Type of Soil	Depth (m)	SPT 'N'$_{60}$	γ'	c-value	δ	Nq	τ_{max}	q_{max}
From E.L	to	E.L				(KN/m^3)	(KN/m^2)	degrees		(Kpa)	(Kpa)
-6.0 m	**to**	**-10.0 m**	**Medium Sand**	**4.0**	**21**	**8.00**	**0.0**	**25**	**20**	**81.3**	**4.8**
-10.0 m	**to**	**-16.5 m**	**Medium Sand**	**6.5**	**25**	**9.00**	**0.0**	**25**	**20**	**81.3**	**4.8**
-16.5 m	**to**	**-18.0 m**	**Silty Clay**	**1.5**	**7**	**9.00**	**40.0**	**0**	**1**	**-**	**-**
-18.0 m	**to**	**-28.0 m**	**Medium Sand**	**10.0**	**27**	**9.60**	**0.0**	**25**	**20**	**81.3**	**4.8**
-28.0 m	**to**	**-36.0 m**	**Dense Sand**	**8.0**	**40**	**10.00**	**0.0**	**30**	**40**	**95.7**	**9.6**

Using API method, the load bearing capacity of pile is presented in Table E-10.1.

Table E-10.1

Calculation of Skin Friction for Pile

Soil Type		LEVEL	Thickness of Layer (M)	Depth in multiple of Dia.	γ	Vertical Pr.	Average Pr. (p_0)	Su	$\psi = c/p_0$	Soil Parameters			$f_s = \alpha * S_u$	$p =$ minimum of $(K.p_o.\tan\delta, \tau_{max})$	Inner Shaft Friction (fsi)	Outer Shaft Friction (fso)
										α	δ	K				
					(KN/m^3)	Kpa	Kpa	Kpa					Kpa	Kpa	(KN)	(KN)
1	Top	-6.00	4.0	4.4 D	8.0	0.0	16.0	0.0	0.0	0	25	1.00	0.00	7.46	84.4	85.9
	Bottom	-10.0				32.0										
2	Top	-10.0	6.5	11.5 D	9.0	32.0	61.3	0.0	0.0	0.00	25	1.00	0.00	28.56	524.9	534.2
	Bottom	-16.5				90.5										
3	Top	-16.5	1.5	13.1 D	9.0	90.5	97.3	40.0	0.4	0.78	0	0.00	31.18	0.00	132.3	134.6
	Bottom	-18.0				104.0										
4	Top	-18.0	6.4	20.0 D	9.5	104.0	134.2	0.0	0.0	0.00	25	1.00	0.00	62.56	1123.2	1143.2
	Bottom	-24.4				164.3										
4	Top	-24.4	3.7	24.0 D	9.5	164.3	164.3	0.0	0.0	0.00	25	1.00	0.00	62.56	645.6	657.1
	Bottom	-28.0				164.3										
5	Top	-28.0	6.0	26.6 D	10.0	164.3	164.3	0.0	0.0	0.00	30	1.00	0.00	77.46	1314.1	1337.4
	Bottom	-34.0				164.3										
														Total	3824.5	3892.5

Calculation of End Bearing and Total Capacity of Pile

Founding Level	Nq	Soil Pr. At pile base σ_v	$q_b = N_q \times \sigma_v$	q_{max}	$q_{des} = \min(q_b, q_{max})$	$Q_{an} = q_{des} * A_n$	$Q_s = q_{des} * A_b$	$Q_t = (Q_{an} + Q_s)$	fsi	$Q_b = \min(Q_t, \Sigma fsi)$	$Q_u = (Q_b + \Sigma fso)$	$Q_{safe} = Q_u / 2.5$
(m)		(KN/m^2)	(KN/m^2)	(KN/m^2)	(KN/m^2)	(KN)	(KN)	(KN)	(KN)	(KN)	(KN)	(KN)
-34.0	40	164.3	6573	9600	6573	150.00	4181.6	4331.6	3824.5	3824.5	7716.9	3086.8

EXAMPLE: 10.4

Using the soil and pile details as given in Example - 10.3, compute the allowable load bearing capacity on pile by applying Gudavalli method. The applicable F.O.S for safe bearing capacity of pile is 2.5.

Solution:

According to Gudavalli method, the skin friction resistance of pile f_s is expressed as; $f_s = \beta . \sigma'_v$

$$\beta = (3.5 - 3.2\ PLR)^{-0.023\ l}\ ;\ PLR = \left(\frac{bi}{1.4}\right)^{0.19}$$

Base resistance of pile is expressed as;

$$q_b = N_q \cdot \sigma' \text{v} \quad ; N_q = 12.3\ PLR^{-8.4}$$

For embedment length of pile (ℓ) = 28.0 m > 20.D

$$PLR = \left(\frac{d_i}{1.4}\right)^{0.19} \quad = \quad 0.919$$

Referring equations in table 10.1 & 10.2 , the value of friction factor β and end bearing capacity factor Nq for driven pile as per Gudavalli method are;

$$\beta = (3.5 - 3.2 . PLR) e^{-0.023 . l} \quad = \quad 0.29$$

$$N_q = 12.3\ x\ PLR^{-8.4} \quad = \quad 24.90$$

The limiting values of skin friction and bearing capacity factor for soil strata is as given below.

Geotechnical Parameters of soil

E.L. of Soil Strata		Type of Soil	Depth (m)	SPT 'N'$_{60}$	γ'	c-value	δ	Nq	τ_{max}	q_{max}
From E.L to	E.L				(KN/m^3)	(KN/m^2)	degrees		(Kpa)	(Kpa)
-6.0 m to	-10.0 m	Medium Sand	4.0	21	8.00	0.0	25		81.3	4.8
-10.0 m to	-16.5 m	Medium Sand	6.5	25	9.00	0.0	25		81.3	4.8
-16.5 m to	-18.0 m	Silty Clay	1.5	7	9.00	40.0	0		-	-
-18.0 m to	-28.0 m	Medium Sand	10.0	27	9.60	0.0	25		81.3	4.8
-28.0 m to	-36.0 m	Dense Sand	8.0	35	10.00	0.0	30	24.9	95.7	9.6

The skin resistance and end bearing of driven pile is presented below in **Table E-10.2**.

Table E-10.2

Calculation of Skin Friction for Pile

Soil Type		LEVEL	Thickness of Layer (M)	Depth in multiple of Dia.	γ	Vertical Pr.	Average Pr. (p_0)	Su	ψ = c/p_0	Soil Parameters		f_s = α *S_u	Inner Shaft Friction (fsi)	Outer Shaft Friction (fso)
										α	β			
					(KN/m^3)	Kpa	Kpa	Kpa				Kpa	(KN)	(KN)
1	Top	-6.00	4.0	4.4 D	8.0	0.0	16.0	0.0	0.0	0	0.29	0.0	53.0	53.9
	Bottom	-10.0				32.0								
2	Top	-10.0	6.5	11.5 D	9.0	32.0	61.3	0.0	0.0	0.00	0.29	0.0	329.7	335.5
	Bottom	-16.5				90.5								
3	Top	-16.5	1.5	13.1 D	9.0	90.5	97.3	40.0	0.4	0.78	0.00	31.2	0.0	0.0
	Bottom	-18.0				104.0								
4	Top	-18.0	6.4	20.0 D	9.5	104.0	134.2	0.0	0.0	0.00	0.29	0.0	705.5	718.0
	Bottom	-24.4				164.3								
4	Top	-24.4	3.7	24.0 D	9.5	164.3	164.3	0.0	0.0	0.00	0.29	0.0	496.7	505.5
	Bottom	-28.0				164.3								
5	Top	-28.0	6.0	26.6 D	10.0	164.3	164.3	0.0	0.0	0.00	0.29	0.0	816.5	831.0
	Bottom	-34.0				164.3								
												Total	2401.3	2444.0

Calculation of End Bearing and Total Capacity of Pile

Founding Level	Nq	Soil Pr. at pile Base σ_v	q_b= N_q x σ_v	Q_{an} = q_{des}*A_n	Q_s = q_{des}*A_s	Q_t = (Q_{an}+Q_s)	fsi	Q_b = min(Q_t, Σfsi)	Q_u = (Q_b+Σfso)	Q_{safe} = Q_u/2.5
(m)		(KN/m^2)	(KN/m^2)	(KN)	(KN)	(KN)	(KN)	(KN)	(KN)	(KN)
-34.0	24.90	164.3	4091.3	93.36	2602.8	2696.12	2401.32	2401.32	4845.32	1938.13

10.5 Concrete Plug for Open Ended Driven Piles

10.5.1 Reinforced Soil Plug

The function of the piles is to transfer the loading from the superstructure to the more compact and less compressible soil- or rock strata. To develop the end bearing resistance in open ended driven piles, soil plug formation is highly desirable. However, the formation of soil plug inside a pile is a complex phenomenon and slippage of soil plug may have an impact on base resistance of pile which is a source of uncertainty. This may cause an excessive settlement in the pile and finally a reduction in end bearing capacity. Based on experience and a conservative design approach, API and other standards suggests empirical correlations to calculate the ultimate axial bearing capacity using partial plug effect or a reduced value of Nq and further limiting the values of skin friction and base resistance of soil as discussed in section 10.4.2. Gudvalli and API etc. suggests quasi-empirical correlations to estimate the load bearing capacity of driven piles is to ensuring the non-slippage of soil plug.

However, to improve the end bearing resistance of an open ended pile, the internal soil column formed inside the pile can be reinforced by placing concrete cap at the top of pile, to form composite plug and derive the full end bearing resistance similar to the closed ended piles. If piles are driven through non-compressible soil strata, using an integrated soil-concrete plug system, full end bearing capacity of pile can be utilised after conformation of field tests as this type of formation is grossly dependent upon construction quality. The non-compressible soil column inside the pipe pile shall be plugged with structural concrete of adequate bond strength to ensure the non-slippage of internal soil column along with soil plug. The concrete plug of length equivalent to 4 to 6 times of pile diameter with structural concrete are fairly reasonable for the purpose. For free standing pile without deck, the cumulative resistance of soil and concrete with in the pile shaft should be estimated. The reinforced concrete plug placed at the top of pile head are generally integrated with superstructure as shown in **Figure 10.7(c)**. Recent field tests have confirmed the behaviour of these piles similar to the conventional close-ended piles.

Furthermore, the structural concrete filling at the top of pile is highly advantageous from durability point of view. In marine environments, corrosion is more severe in splash zone and this is the pile segments where maximum bending and shear stress are induced due to lateral loads. In long run, during residual life of structure, this section is highly stressed due to corrosion loss of steel and reduction of thickness as well as reduction of section modulus of pile. The concrete infill inside the pipe pile form composite section to increases the section modulus and rigidity of piles and even after considerable loss of steel section, the minimum required integrity of the pile can be maintained with the substrate.

Figure 10.7(A) Indicates the pipe pile foundation dependent on soil plug only. Whereas, **Figure 10.7(B)** and **Figure 10.7(C)** schematically indicates the pile with reinforced soil plug (piles with compacted soil fill and concrete capping). If piles are driven through the soft or loose soil strata, concrete plug should be placed at base of pile as discussed in following section.

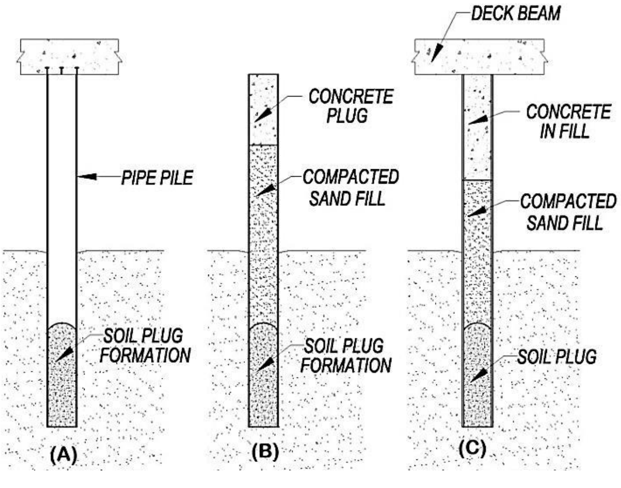


Figure 10.7 Conditions of Soil-plug in Driven Pipe Piles

10.5.2 Formation of concrete Plug at the Base of Pile

In offshore situations, generally the pile is driven through water, through compressible soil to the bearing sand- or rock layer. If pile is driven through predominantly loose sand or compressible clay layer to the refusal, the chances for formation of soil plug at the bottom of pile is very remote. Then the end bearing resistance of pile is mainly governed the point of minimum cross-section i.e., annulus of pile. To achieve maximum end bearing resistance, the loose or compressible soil which entered inside the pile is removed by jetting and air lifting or by drilling and well compacted cement concrete mass or grout is placed to form plug at the pile base. The end bearing resistance due to concrete plug mainly depends on the bond strength between inner surface of pipe pile and concrete plug formation.

There are two possible methods of transferring the axial force between the plug and the pile. One method is by the natural bonding between the two materials and the second with the help of shear keys. Here, the details of natural bond resistance between the pipe pile and concrete plug is briefly discussed. For design of shear keys, other references should be referred. In the absence of reliable comprehensive data for connection strength, the allowable axial load transfer (through concrete plug) should be taken as the smaller value (pile or sleeve) of the force calculated by a multiplication of the contact area between the concrete plug and steel surfaces and the allowable axial load transfer stress f_{ba}. Where, f_{ba} should be an appropriate value for the concrete/steel interface.

The Eurocode prescribes a constant value of 0.55N/mm^2 for bond in circular hollow sections (ECS (2011d)). So, despite of the pile diameter, shrinkage properties, pile wall thickness, surface roughness, concrete strength, compaction levels, additives or plug length, this value could be used in design. The American Petroleum Institute (API) prescribes a value of f_{ba} as 0.138 MPa and 0.184MPa for design of piles in normal condition and extreme condition respectively. The British Standard (BSI (1999)) adopts a value of 0.4N/mm^2 for concrete filled hollow steel sections.

If inner surface of pile shaft is clean from fines or clay to ensure the proper bonding between the concrete and steel, and properly compacted concrete is to be placed, higher bond resistance between steel and concrete may be adopted. However, for removal of loose or soft soil from deep pile, if boring or jetting is to used and concrete is to be placed under the water using tremie then lower bond friction shall be adopted.

10.6 Static Axial Resistance in Rock

The basis of the 'soil mechanics approach' to calculate the axial load carrying capacity of pile is its ability to resist the load in compression and while a pile bearing on rock or some other hard incompressible material, it is known as an end-bearing pile. For maximum economy in the cross-sectional area of a pile it is desirable to drive the pile to virtual refusal on a strong rock stratum, to develop its maximum carrying capacity. The load bearing capacity of ground at pile base depends on the representative shear strength parameters of the concerned rock strata. These are not necessarily minimum or average values, but the parameters selected by the engineer using his experience and judgement and taking into account the variability in the geological conditions, the number of test results available, the care used in taking samples and selecting them for test, and experience of other site investigations and of the behaviour of existing structures in the locality.

If a pile is driven to near refusal on to a strong *intact* rock the safe working load on the pile is governed by the permissible working stress on the material of the pile at the point of minimum cross-section. The ability of the rock to sustain this loading without yielding depends partly on the compressive strength of the rock and partly on the frequency and inclination of fissures and joints in the rock mass, and whether these discontinuities are tightly closed or are open or filled with weathered material. Kulhawy and Goodman showed that the ultimate base resistance (q_{ub}) can be related to the RQD of the rock mass as shown in Table- 10.4.

Table-10.4: Rock mass properties (Kulhaway and Goodman, 1988)

RQD (%)	$\boldsymbol{q_{ub}}$	$\boldsymbol{c}$	*ϕ (°)*
0 - 70	$0.33\sigma_c$	$0.1\sigma_c$	30
70 - 100	$0.33 - 0.8\sigma_c$	$0.1\sigma_c$	30 - 60

- *σ_c = UCS = Uniaxial Compressive Strength of intact rock core.*

Where laboratory tests can be made on undisturbed samples of weak rocks to obtain the parameters *c* and ϕ. Kulhawy and Goodman state that the ultimate bearing capacity of the jointed rock beneath the pile toe can be obtained from the equation:

$$q_{ub} = cN_c + \gamma D \cdot N_q + 0.5 \cdot \gamma \cdot B \cdot N_\gamma \tag{10.16}$$

For rock strata the Bearing capacity factors shall be evaluated from **Figure 10.8**.

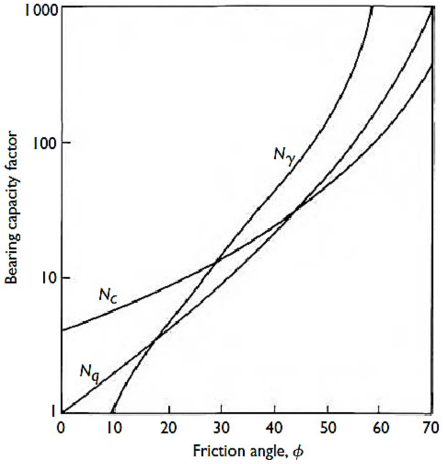


Figure 10.8 Wedge bearing capacity factors for foundations on rock (Pells and Turner)

The above equation is appropriate for rock strata exist with soil overburden and generally the fixidity of pile is being governed by overlaying soil strata. Overburden soil mass provides stability to fractured rocks against dislocations. For general requirements, the compressive strength of rock can be evaluated as explained below.

10.1.1Compressive Strength of Rock

The correlation between the intact rock compressive strength (σ_c) and RQD-corrected rock mass compressive strength (σ_{cm}) can be expressed by the following equation.

$$\sigma_{cm} = \alpha_E \text{ x } \sigma_c \tag{10.17}$$

Where: σ_{cm} = corrected rock mass compressive strength

α_E = reduction factor

= $10^{(0.013(RQD)-1.34)}$

RQD = Rock Quality Designation

σ_c = unconfined compressive strength of intact rock

The reduction factor for this analysis based on recommendation by Kulhawy and Goodman (1987), suggested that a first approximation of rock mass strength be taken as $0.33\ \sigma_c$, when RQD less than about 70% and the linearly increasing to $0.8\sigma_c$ when RQD increases from 70% to 100%.

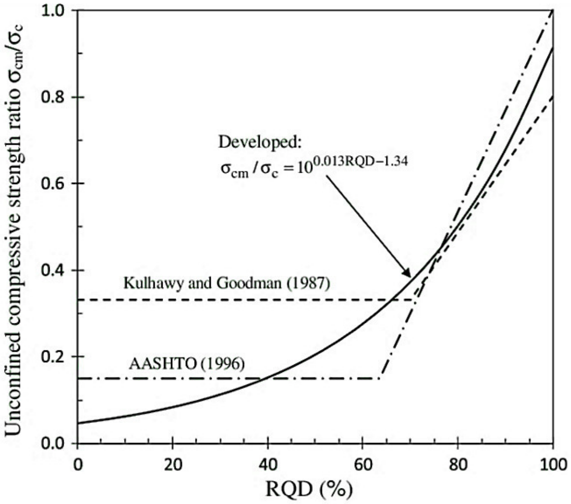


Figure 10.9 Comparison of $^{\sigma_{cm}}/_{\sigma_c}$ versus RQD relationships

The Standard Specification for Highway Bridges adopted by the American Association of State Highway and Transportation Officials (AASHTO 1996) suggest that σ_{cm} be estimated using the following equation:

$$\sigma_{cm} = \sigma_E \text{ x } \sigma_c \tag{10.18}$$

$$\sigma_E = 0.0231\ (RQD) - 1.34\ \geq 0.15 \tag{10.19}$$

Zhang and Einstain (2004) developed a nonlinear variation of $^{\sigma_{cm}}/_{\sigma_c}$ with RQD as shown in **Figure 10.9**, which covers the entire range of RQD from zero to 100 percent.

Example: 10.5

A 12.5 m long steel pipe pile (Do = 618 mm, Di = 600 mm) driven through 12m thick loose sandy strata and socketed into the weathered rock strata. The levels and properties of soil strata are given below;

Soil strata

i) ***EL. 3.0M to -9.0M***

Bulk density of soil (γ) = ***17.00*** *KN/m^3*

Shear parameter of soil (φ) = ***28.00*** *deg,*

(c) = ***0.00*** *KN/m^2*

ii) ***EL. -9.0M to -20.0M***

Compressive strength of rock (q_c) = ***3500*** *KN/m^2*

RQD of rock = ***60.00*** *%*

Bulk density of rock = ***22.00*** *KN/m^3*

Level of Ground water table = ***(+) 1.0*** *M*

Compute the allowable load on pile with concrete plug at the base and a factor of safety as 6 against shear failure for rock.

Solution:

Length of pile (L) = **12.0** M

Outer surface area of pile /m run = **1.94** m^2/m run

Area of pile annulus (An) = **0.017** m^2

Total area at pile base (Ap) = **0.283** m^2

Estimation of Skin Friction of Pile

a) Skin friction of pile for (segment - 1)

From EL. 3.0M to 1.0M (above water table)

Undrained shear strength; Su = **0.0** KN/m^2

Effective unit weight of soil (γ') = **17.0** KN/m^3

Vertical stress at top of strata (p_{r1}) = **0.0** KN/m^2

Vertical stress at bottom (p_{r2}) = **34.0** KN/m^2

σ_v

Mean vertical effective stress = **17.0** KN/m^2

Referring equation in Table-10.1;

K = **1.0**

δ = **20** deg.

β = K.tanδ β = **0.36**

$f_s = \beta.\sigma_v$ f_s = **6.19** KN/m^2

< **2.9** Mpa; $\boldsymbol{\tau_{max}}$ **(O.K)**

Length of pile segment (L_1) = **2.00** m

Outer surface area of pile $A_{so} = (\pi \times D_o \times L_1)$ = **3.88** m^2

Skin friction of pile $f_s.A_{so}$ = **24.03** KN

b) Skin friction of pile for (segment - 2)

From EL. 1.0M to -10.0M

Undrained shear strength; Su1 = **0.0** KN/m^2

Effective unit weight of soil (γ') = **7.0** KN/m^3

Vertical stress at top of strata (p_{r1}) = **34.0** KN/m^2

Vertical stress at bottom (p_{r2}) = **111.0** KN/m^2

Mean vertical effective stress = **72.5** KN/m^2

Referring equation in Table-10.1;

K = **1.0**

δ = **20.0** deg.

β = K.tanδ β = **0.36**

$f_s = \beta.\sigma_v$ = **23.06** KN/m^2

Length of pile segment (L_2) = **11.00** m

Skin friction for (segment - 2) $f_s.A_{so}$ = **563.55** KN

$\boldsymbol{\Sigma f_s.A_{so}}$ = **587.58** KN

Estimation of End Bearing Capacity of Pile

For weathered rock

q_c = **3500** KN/m^3

$c = 0.1 \cdot q_c$ c = **350** KN/m^3

for RQD = 60 % φ = **30** deg

Considering the plugging effect in pile

Area at pile base (Ap) = **0.30** m^2

Pile resistance at base

$$q_{ub} = cN_c + \gamma D \cdot N_q + 0.5 \cdot \gamma \cdot B \cdot N_\gamma$$

c = **350.0** KN/m^2

Bearing Capacity Factor for ϕ = 30 deg

Nc = **15.0**

Nq = **9.0**

N_γ = **15.0**

σ_v = **111.0** KN/m^2

c.N_c = **5250.0** KN/m^2

$\gamma.D.N_q$ = **999.0** KN/m^2

0.5,γ.B. N_γ = **55.6** KN/m^2

Qub = **6304.62** **KN**

F.O.S adopted for S.B.C of Pile = **6**

Safe Bearing Capacity of pile = **1050.77** KN

Design of Concrete Plug

Inner perimeter of pile = **1885.0** mm

Frictional resistance between pile and concrete = **0.134** MPa

Resistance of concrete plug per meter length = **252.6** KN

Length of concrete plug at base = **4.1** m

Chapter 11

Drivability of Piles for construction

The engineer is a mediator between the philosopher and the working mechanic, and the absolute necessity is of possessing both practical and theoretical knowledge

- Henry Palmer

11.1 Introduction

Pile drivability refers to the ability of a pile to be driven to a desired penetration depth and/or capacity. The drivability of a pile to the designed depth is very important and must be evaluated during design. If pile cannot be driven to the required design depth or up to the ultimate capacity without sustaining damage, all of the previously discussed static analysis and design methods are meaningless. The pile must have sufficient drivability to overcome the soil resistance encountered during driving to reach the estimated or specified pile penetration depth. The limit of pile drivability is the maximum soil resistance a pile can overcome without sustaining damage or a refusal driving resistance with a properly sized driving hammer.

If a pile section does not have a drivability limit in excess to overcome the soil resistance during driving, it will not be driveable to the desired penetration depth. The failure to adequately evaluate pile drivability is one of the most common deficiencies in driven pile design practice

11.2 Factors Affecting Drivability

The drivability of pile largely depends on following two aspects;

- First, the pile must have sufficient stiffness to transmit the large enough driving forces to overcome soil resistance.

- Second, the pile must have sufficient structural strength to withstand the driving forces without damage and total stress applied on the pile should be within permissible stress limit.

Generally, the highest stress levels occur in a pile during driving as high driving stresses are necessary to cause pile penetration. In order to drive the pile up to the design depth, the pile must be stressed to overcome the ultimate soil resistance, plus other dynamic resistance forces. During pile driving, the high strain energy and temporary nature of the loading allow a substantially higher driving stress in comparison to that of static design case.

11.2.1 Pile Driving System

The fundamental pile driving system consists of the pile; the pile driving hammer, the soil and the other associated auxiliaries but very significant are the crane, crane boom leads and the template as discussed below;

11.2.2 Pile

A driven pile should remain within structural and buckling stress limits under the dynamic, or driving induced loads as well as under the static loading conditions during its service period. Therefore, the material stress limits are placed on:

1. The maximum allowable driving stresses,
2. The maximum allowable design stress during the service life,

For a satisfactory driving, the pile stresses during driving shall be within allowable stress limit. For steel piles, the allowable maximum compressive stress during driving shall be within 90% of the minimum yield strength of the pile material.

11.3 Pile Driving Hammers

Driven Steel piles or sheet piles are generally installed by a special mechanical device known as a pile hammer. The graphic in **Figure 11-1**, presents the basic components of a pile driving hammer. During execution, the ram impacts the anvil and striker plate at a certain kinetic energy. This impact energy is transferred through the hammer cushion (also called cap-block), then onto the helmet (also called pile cap) and finally the pile to overcome soil resistances.

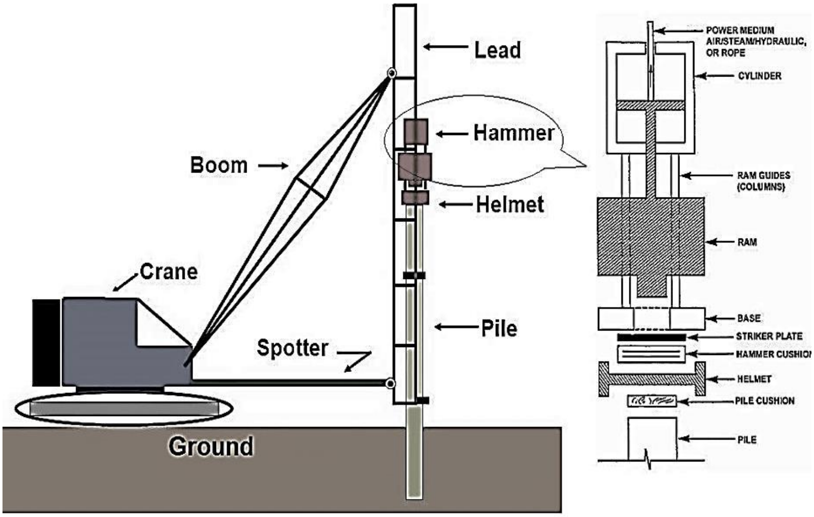

Lead
Hammer
Boom
Helmet
Crane
Spotter
Pile
Ground
POWER MEDIUM AIR/STEAM/HYDRAULIC, OR ROPE
CYLINDER
RAM GUIDES (COLUMNS)
RAM
BASE
STRIKER PLATE
HAMMER CUSHIO
HELMET
PILE CUSHION
PILE

Figure 11.1 Schematic Arrangement of (Impact type) Pile Driving Hammer

The common types of hammer used for pile driving are:

a) Air or Steam hammers,

b) Diesel hammers: (These could be Open end Diesel hammers or Closed end Diesel Hammers.),

c) Hydraulic hammers, and

d) Vibratory hammers

The first three types of hammers are impact hammers, meaning they drive the piles into the ground by impacts or blows. The most popular type of hammer is the open-end Diesel hammer. The fourth one, the vibratory hammers, uses continuous vibratory forces to push the pile into the ground. This hammer is commonly used on piles or steel sheet piles. Vibratory hammers are also used to extract the piles.

The hammer proposed for installation of piles must be able to provide variable energy. This means the hammer should have variable strokes that can be adjusted and must be able to drive the piles at reasonable blow counts without overstressing or damaging the piles. Generally, the driving equipment must provide the required energy at a blow count ranging from 1 blow per centimetre (approx. 36 blows per foot) to 4 blows per centimetre (approx. 120 blows per foot) at the end of initial drive. This range of blows per centimetre allows the hammer to be properly sized. If the hammer mobilizes the capacity with very few blows the hammer is too big and may damage the pile. If, on the other hand, a hammer needs too many blows, the hammer is too small and we may risk the capacity of not being properly mobilized.

Here, the term “***stroke***” refers to the height of the hammer ram that falls to hit the pile, “***bounce chamber pressure***” refers to a device typically used in double acting Diesel hammers discussed later. In certain cases, such as hydraulic hammers, to determine the impact energy correctly, it is necessary to install a device to measure the velocity of the ram just before impact.

11.3.1 Single Acting Air Hammer

In a single acting hammer, a heavy ram is lifted up by compressed air but dropped by its own weight. The energy of a single acting hammer is equal to the weight of the ram times the height of fall.

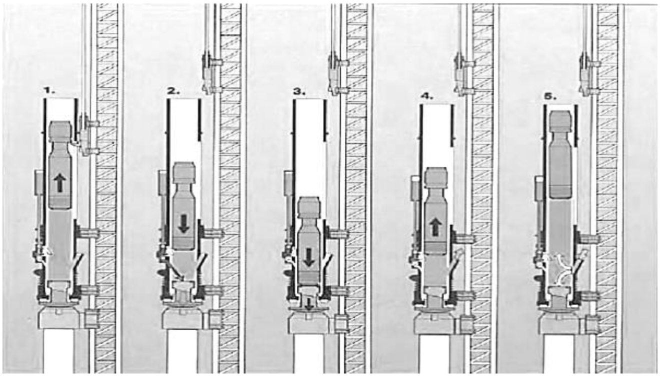


Figure 11.2 Working Principle of Single Acting Hammer

They are advantageous when driving heavy piles in compact or hard soils; the heavy ram striking at - low velocity produces least damage due to impact. The image shown below represents the working principle of an air/steam single acting hammer. The advantages of single acting air hammer are:

- Apply same stroke at each impact
- Have consistent operation rate
- Low impact velocity
- More efficient than diesel
- Cleaner exhaust than diesel

The disadvantages of this hammer are:

- Additional support equipment required
- Heaviest hammer
- Not as dependable as diesel
- Thick hammer cushion stack required

Single acting air/steam hammers are essentially gravity, or drop hammers, in which the hoist line is replaced by a pressurized medium, being either steam or air. While originally developed for steam power, most of these hammers today operate on compressed air. To lift the ram weight with motive pressure, a simple one-cylinder steam engine principle is used. During the upstroke cycle, the ram is raised by externally produced air or steam pressure acting against a piston housed in the hammer cylinder. The piston, in turn, is connected to the ram by a rod. During the down-stroke cycle, the ram falls by gravity (less friction) to impact the striker plate and hammer cushion.

The maximum stroke of single acting air/steam hammers generally ranges around 60 centimetres to 1.5 meter. Single acting air/steam hammers have the advantages of moderate cost and relatively simple operation and maintenance. They are versatile for many pile types, particularly large concrete and steel pipe piles.

11.3.2 Single Acting Diesel Hammers

The basic distinction between all diesel hammers and all air/steam hammers or hydraulic is that, whereas air/steam hammers are one-cylinder engines requiring motive power from an external source, diesel hammers carry their own fuel from which they generate power internally. The initial power to lift the ram must be furnished by a hoist line or other source to lift the ram upward on a trip block. After the trip mechanism is released, the ram guided by the outer hammer cylinder falls under gravity. As the ram falls, diesel fuel is injected into the cylinder below the air/exhaust ports. Once the ram passes the air/exhaust ports the diesel fuel is compressed and heats the entrapped air. As the ram impacts the anvil, the fuel explodes, increasing the gas pressure. The combination of ram impact and fuel explosion drives the pile downward, and the gas pressure and pile rebound propels the ram upward in the cylinder. On the upstroke, the ram passes the air ports and the spent gases are exhausted. Since the ram has a velocity at that time, the ram continues upward against gravity, and fresh air is pulled into the cylinder. The cycle then repeats until the fuel input is interrupted.

The Diesel hammers should also have variable energy settings, at least three fuel settings that will produce reduced strokes. The hammer should be operated and maintained within the manufacturer’s specified ranges.

11.3.3 Double acting or Close-End Diesel Hammers

The double acting diesel hammer works very much in principle like the single acting diesel hammer. The main change consists of a closed cylinder top. When the ram moves upward, air is being compressed at the top of the ram in the so called "bounce chamber" which causes a shorter stroke and therefore a higher blow rate. Operationally, as the ram passes the bounce chamber port and moves toward the cylinder top, it creates a pressure which effectively reduces the stroke and stores energy, which in turn will be used on the down-stroke. Like the single acting hammer, the actual stroke depends on fuel charge, pile length and stiffness, soil resistance, and condition of piston rings. As the stroke increases, the chamber pressure also increases until the total upward force is in balance with the weight of the cylinder itself.

This type of hammer has the following advantages:

- No additional support equipment required,
- Drive piles faster, and it has lightweight.

This hammer also has the following disadvantages:

- Lowest efficiency and the Most difficult to spot operation problems.

11.3.4 Hydraulic Hammers

Hydraulic hammers are a modern type of piling hammer used instead of diesel and air hammers for driving steel piles.

Hydraulic hammers have the following advantages:

- Controllable variable stroke, High efficiency blow,
- Low impact velocity (Short strokes),
- Wide range of sizes available,
- Clean running/quiet.

Hydraulic hammers have the following disadvantages:

- Need hydraulic power pack and hoses,
- Need dedicated person for hydraulic controls,
- Reparability / highly qualified tech required, and
- Expertise in hammer operation is needed.

For effective installation of piles, variable energy hydraulic hammers with least three hydraulic control settings with predictable energy or equivalent ram stroke are preferred. The shortest stroke should be a maximum of 2 feet for the driving of concrete piles. The remaining strokes shall include full stroke and approximately halfway between minimum and maximum stroke.

11.3.5 Vibratory hammers

These hammers are generally used for driving and extracting sheet piles and non-displacement H-piles and pipe piles. They are not impact hammers. Vibratory hammers use paired counter-rotating eccentric weights to impart a sinusoidal vibrating axial force to the pile (the horizontal components of the paired eccentric weights cancel). Most common hammers operate at about 1000 Hz. However, the recently introduced hammers have frequency up to 3000 r.p.m.

These hammers are rigidly connected by hydraulic clamps to the pile head and may be used for either pile installation or extraction. Vibratory hammers are not rated by impact energy delivered per blow, but instead are classified by energy developed per second and/or by the driving force they deliver to the pile. The power source to operate it is usually a hydraulic power pack. Vibratory hammers of sufficient capacity (force and amplitude) should be used to drive steel sheet piles.

However, it should be noted that, currently there is not a method to establish the bearing capacity of pile with a vibratory hammer. Therefore, to accept piles from the bearing point of view, a blow count criterion will be required and therefore driving using an impact hammer will be required at the end of the driving.

11.4 Other Accessories for Pile Driving System

11.4.1 Cushion

Most hammers use a hammer cushion between the hammer and the helmet to relieve the impact shock, thus protecting the pile hammer. However, some hammer models exist that do not require a hammer cushion, or utilize a direct drive option where the hammer cushion is replaced by a steel striker plate. Cushions are used to protect both the hammer and the pile. Cushions that protect the hammer are called capblock cushions or hammer cushions. Hammer cushions are constructed of man-made materials that are heat resistant, durable and absorb a certain amount of shock. Hammer cushions are used on all impact hammers except gravity (drop) hammers.

11.4.2 Pile Helmet

Pile helmet should be suitable for the type and size of piling being driven and contain the required thickness of pile cushion and to assist in maintaining pile-hammer alignment. Use a pile helmet that fits loosely over the pile head and is at least 1 inch larger than the pile dimensions and properly designed so that it will not restrain the pile from rotating. The main concept to retain here is that the helmet must be one inch larger than the pile dimension.

11.4.3 Crane

The crane must be large enough to handle the leads, hammer and the pile. Cranes are usually rated based on their lifting capacity. For example, a small pile driving job requires a 50 to 100 ton crane. Whereas, for larger jobs and waterfront work, heavier cranes up to 250 tons capacity or more may be required. The crane's boom should be long enough to reach about 10 meters above the longest pile due to the height of the hammer and leads above the pile top. Longer booms are also required for special site conditions such as pitching of flat web sheet piles for cellular cells. The boom angle is proportional to the cranes lifting capacity. The greater the angle (from the vertical), lower the lifting capacity.

11.4.4 Leads

Leads are used to align the pile and hammer during driving. There are numerous different lead designs. Lead systems include three main types: swinging, semi-fixed, and fixed leads, schematically as shown in **Figure 11-3**.

i. **Swinging Leads**: These types of leads are suspended from the crane line and can hang freely. There is not fixity at the bottom and at the top of the lead. When this type of leads is used, template must be used.

ii. **Semi-fixed Leads**: These types of leads have some restriction at the bottom. However, it is not fixed at the top and the lead can swing in one direction. When this type of leads is used, template is also required.

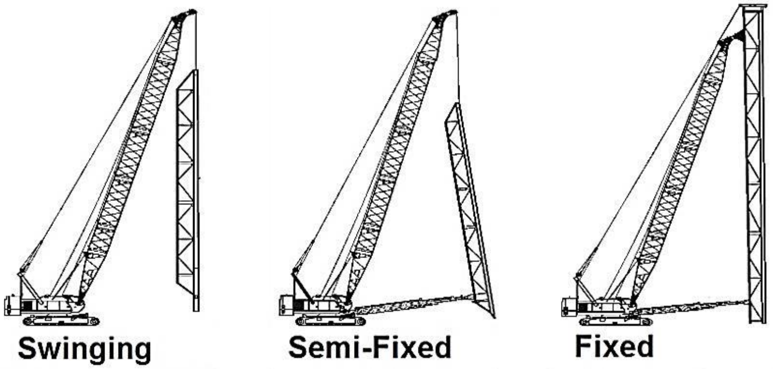


Figure 11.3 Basic types of leads for pile driving

iii. **Fixed Leads**: Fixed Leads are fixed at the top and at the bottom. When this type of lead is used, the template is not required.

Leads must provide a freedom of movement to the hammer and that have the strength and rigidity to hold the hammer and pile in the correct position and alignment during driving.

11.4.5 Templates

Templates are required to ensure the piles are placed at the right location and the correct alignment. Templates are not required where fixed leads are utilized. When positioning templates that include batter piles, it must be remembered that the correct template position of batter piles will vary depending upon the template elevation relative to the pile cut-off elevation. For example, consider a template located 1.6 meter above the pile cut-off elevation. If the plan pile locations at cut-off are used at the template elevation, a 1H:4V batter pile would be 40 centimetres out of location at the pile cut-off elevation. Template construction should also allow the pile to pass freely through the template without binding. Templates with rollers are preferable, particularly for batter piles.

11.4.6 Followers

Followers are structural members to assist in driving, when the top of the pile is below the reach of the hammer. For example, when the pile head is below the water or in a special situation a pile needs to be driven below the deck of an existing structure.

Followers are constructed of steel with an adequate cross-section to withstand driving stresses. For driving steel piles, the cross-sectional area of the follower should be greater than or equal to the cross-sectional area of the pile.

A pile helmet may be attached at the lower end of the follower according to the requirements of fittings.

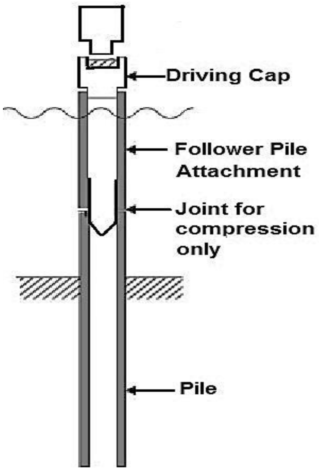


Figure 11.4 Schematic Arrangement of Follower Pile

11.5 Analysis of Soil to Check the Pile Drivability

For successful driving of sheet pilings, it is essential that a good knowledge of the site conditions be acquired in order to enable an assessment of the topographical and sub-soil conditions before actual installation. The practical information shall always be considered to ensure the proper installation of sheet piles.

Each site is subject to its own unique set of restrictions, which may vary according to the proximity and nature of neighbouring features. The soil is the medium which must be penetrated by the pile element. In order to achieve the required penetration of the pilings or sheet pilings, a site investigation of the soils, coupled with various field and laboratory tests, can greatly aid to the installation by providing relevant information on the followings;

a) Stratification and hardness of the subsoil strata;
b) Particle shape and size distribution; fine contents;
c) Consistency limits;
d) Density of sub-soil;
e) Existence of groundwater table;
f) Shear parameters;

11.5.1 Stratification and Hardness of the Subsoil Strata

The stratification of various sub-soil strata and their consistency provide substantial information for qualitative analysis and selection of pile driving system.

- Easy driving conditions are to be expected in soft or very soft soils such as peat, silt, marine clay, etc. Easy driving conditions are also to be generally expected in loosely bedded medium and coarse sands and gravels with no rock inclusions, unless there are embedded cemented strata.
- Moderate driving conditions are to be expected in moderately dense bedded medium and coarse sands, fine gravel soils and firm clays and loams.
- Difficult to very difficult driving conditions are to be expected in most instances of densely bedded medium and coarse gravels, densely bedded fine sands and silty soils, embedded cemented strata, stiff to very stiff clays, cobbles and weathered or soft to medium-hard rock.

The stratification and stiffness of soil can be assessed from bore-log profile and SPT test results.

11.5.2 Particle Shape and Size Distribution, Fine Contents

For granular soil, grain size distribution and effective size of particles provides the information related to permeability of sub-soil and condition of effective stress during driving.

Vibratory driving is particularly successful in sands and gravels with a rounded grain shape and in very soft or soft soil types with low plasticity. Vibrating is much less suitable for highly cohesive soils or sands and gravels with an angular grain shape. Dry fine sands and firm marl and clay soils are particularly critical as they absorb the energy of the vibrator without reducing the skin friction and base resistance. If the subsoil is compacted during vibration, then its penetration resistance can increase to such an extent that the pile being installed can no longer reach the required depth. This risk arises, in particular, when piles and sheet piles are installed at close spacing and when using vibration in non-cohesive soils. Vibratory driving must be stopped in such cases.

When plastic fines content (plastic clay or silt) are greater than 15%, it usually dictates that the use of vibratory-drivers will not be very effective.

11.5.3 Consistency Limits

The Atterberg limits indices are used for classifications of cohesive soils. They provide information for interpreting several mechanical and physical properties of clayey soil. High plasticity index means more clayey in nature. When a pile penetrates into a high plastic clay deposit, causes large shear strains in the surrounding clay, and as a consequence induce large changes in the total stress and pore water pressure in the regime. Driving of displacement piles into such soil generate excess pore pressures close to the pile and at the pile-soil interface. In such cases, pile set-up and relaxation characteristics should be carefully studied at site. Over-consolidated plastic clay strata experiences very high pore water pressure and set-up effect. The displacement of cohesive soil at the penetrating pile tip will cause remoulding and create excess pore water pressures. Remoulding is a process where the clay particles are reoriented parallel to the shearing surface. Due to remoulding and the excess pore water pressure the soil shear strength reduces. Excess pore water pressure is generally observed in over-consolidated clays; in normally consolidated clays the drop-in strength is entirely due to particle reorientation.

11.5.4 Density of Soil

The driving resistance of pile has been found to increase with increasing relative density. The volume changes (densification or dilation) induced in the soil has been found to relate to a characteristic driving resistance. Soil with high relative density causes more confining pressures to the pile-driving-medium to increase the driving resistance. While vibratory driving, the liquefaction resistance of soil also increases with increasing confining stress of the soil.

11.5.5 Level of Groundwater Table

Moist or dry soils present a greater resistance to penetration during impact driving than those in submerged condition. Driving resistance is increased for a lower degree of saturation due to the lack of liquefaction. This does not apply to saturated cohesive soils, and silts.

11.5.6 Shear Parameters

The shear strength of soil gives the information about skin friction and end bearing resistance of piles at various depths. It provides very significant information in case of granular soil or clayey deposits; however, the shear parameters have only a limited significance when it comes to describing the behaviour of the weathered or disintegrated rock etc. during the installation of piles and sheet piles. For example, rocky calcareous marl can exhibit relatively low shear parameters due to its fissuring, but may in fact present difficult conditions in terms of piling.

11.6 Suitable Driving System for Various Soil

The different types of soil present a variety of driving characteristics. A brief description of each is given below;

11.6.1 Impact Driving

Diesel impact hammers perform particularly well in cohesive or very dense non-cohesive soil layers. Under typical site conditions, it is normal to select a ratio of drive-weight to weight-of-pile plus cap of a 1:2 to 1.5:1 ratio. In accordance with the hammer manufacturer's recommendations, a penetration of 25mm (1 inch) per 10 blows should be considered the limit for the use of diesel hammers. Under certain circumstances, a penetration of 1mm (0.04 inch) per blow could be allowed for a short period of time near the refusal of pile. Longer periods of time at this blow rate would cause damage to both the equipment and the pile.

11.6.2 Vibratory Driving

Round-grain sand or gravel and soft soils are especially suited to vibratory driving. Angular- grain material, or soils with firm consistency, is much less suited. Also, dry soils give greater penetration resistance than those which are either moist or submerged or fully saturated. Based on past experiences of piling contractors, following details may be referred for selection of driving equipment.

Table - 11.1: Driving Methods Suitable for Cohesionless Deposits

SPT value	Vibro Drive	Impact Drive	Pressing in Singles (with Jetting)
0 - 10	Very Easy	Suitable with small drop	-
11 - 20	Easy	Easy	Suitable
21 - 30	Suitable	Suitable	Suitable
31 - 40	Suitable	Suitable	Suitable with pre-auger
41 - 50	Very Difficult (suitable with large vibro-hammers only)	Suitable for high modulus piles with large drop hammers	Suitable with pre-auger
< 50	Not Recommended	-Do-	Very Difficult

Table - 11.2: Driving Methods Suitable for Cohesive Deposits

SPT value	Vibro Drive	Impact Drive	Pressing in Singles (with Jetting)
0 - 15	Easy	Suitable with small drop	-
16 - 25	Suitable	Suitable with small drop	Easy
26 - 50	Suitable but less effective with depth (Use large vibro-hammers)	Suitable with large drop	Easy
51 - 75	Very Difficult	Suitable for high modulus piles with large drop hammers	Suitable
76 - 100	Not Recommended	-Do-	Suitable
< 100	Not Recommended	-Do-	Difficult

If the granular sub-soil is further compacted by the vibrations, then penetration resistance will increase sharply, thereby causing refusal.

11.7 Estimation of Pile Driving capacity

Many attempts have been made in the past to determine the relationship between the dynamic resistance of a pile during driving and static load carrying capacity of the pile. The reason is obvious, there is a practical need for a dynamic formula. The majority of pile foundations are installed in soil profiles having variable nature. In such cases, the only convenient method for determining whether or not a pile has reached sufficiently dense strata is by observing the ease or difficulty with which the pile penetrates the ground. At present, a variety of dynamic pile-driving formulas are being used for this purpose in code of practice for foundations and specifications. Most of the dynamic pile-driving formulas are based on the fundamental of energy equation. Wherein the kinetic energy of the hammer's ram at impact is equated to the work done on the pile, that is, the product of the distance the pile moves and the soil force resisting this movement. The fundamental assumption made in all dynamic formulae is that the resistance of piles to further penetration under the permanent load has a direct relationship to their resistance to the impact of the hammer at the time of driving. Dynamic formulae may give reasonably accurate results in permeable and non-cohesive deposits, which allow dissipating the pore water pressure easily and therefore do not present a substantially different resistance to the impact forces during driving the subsequent static loads.

Dynamic formulae are not very accurate to cohesive deposits such as saturated silts, muds, and clays. In these soils the resistance to impact at the toe of the pile is exaggerated by their low permeability, while the frictional resistance on its sides during driving is reduced by remoulding. The tendency of the ground to alter its resistance after driving, which is usual with the fine-grained soils, should be ascertained by re-driving (re-striking) the test piles and an occasional working pile after a period of rest. Load tests on such piles immediately after driving may be misleading, and it is preferable to allow them a stipulated rest time before the load test.

11.7.1 Dynamic pile Formulae

As discussed above, these are based on the laws of dynamic impact on elastic bodies. They correlate the energy of the hammer blow to the work done in overcoming the resistance of the ground to the penetration of the pile. ***The relationship between the dynamic resistance of a pile during driving and static load carrying capacity of the pile are called pile driving formula*** and have been established theoretically and empirically. Of many pile driving formulae which have been proposed, Engineering News Formula, Hiley Formula and Janbu Formula are widely used. Among these the Hiley Formula and Janbu Formula are convenient to use and give reasonable predictions of ultimate bearing capacity of driven piles in granular soils.

11.7.2 Engineering News Formula

According to the Engineering News formula the allowable load of driven pile is given by:

$$Q_a = \frac{Wh}{F\langle S+C\rangle} \tag{11.1}$$

Where, Q = Allowable load in kg

W = Weight of hammer

h = Height of fall in cms

s = Final settlement per blow known as set

c = Empirical constant (c = 2.5 cm for drop hammer and 0.25 cm for single acting and double acting hammer)

F = Factor of safety (Usually taken as 6)

11.7.3 Janbu Formula

The relation suggested by Janbu for ultimate capacity of the pile is:

$$Q_u = \frac{\eta Wh}{s} \times \frac{1}{K_u} \tag{11.2}$$

Where,

$$K_u = C_d\left[1 + \sqrt{\left(1 + \frac{\lambda_c}{C_d}\right)}\right] \tag{11.3}$$

$$C_d = 0.75 + 0.15\frac{Wp}{W} \tag{11.4}$$

$$\lambda_c = \frac{\eta.W.h.L.}{AEs^2} \tag{11.5}$$

Where, = C_d, λ_c Empirical constants

h = Height of fall

W = Weight of hammer

Wp = Weight of pile

E = Modulus of elasticity of pile material

s = Final set of hammer

η = Efficiency factor

The efficiency factor is dependent on the pile driving equipment, the driving procedure adopted, the type of pile and the ground conditions. Values of η can be chosen as follows.

η = 0.70 for good driving conditions

η = 0.55 for average driving conditions

η = 0.40 for difficult or bad driving conditions

11.7.4 Hiley Formula

The relation suggested by c for ultimate bearing capacity of the pile (Q) is:

$$Q = (K\eta Wh)/(S + C/2) \qquad (11.6)$$

Where, K = Hammer coefficient

C = Sum of elastic compression (C = C_p + C_c + C_q)

C_p = Compression of pile

C_c = Compression of pile head

C_q = Compression of ground

W = Weight of hammer

K = 1.0 for trigger-operated drop hammers,

= 0.80 for normally proportioned winch-operated drop hammers;

= 0.90 for single-acting hammers and McKiernan-Terry type of double-acting hammers,

(The hammer should be operated at its maximum speed whilst the set is being taken.)

Q is the ultimate driving resistance in kN

W is the weight of the ram in kN.

h is the height of the free fall of the ram or hammer (in mm),

S is the final set or penetration per blow in millimetres.

C is the sum of the temporary elastic compressions (in mm) of the pile, dolly, packing, and ground, calculated or measured as prescribed below.

η is the efficiency of the blow, representing the ratio of energy after impact to striking energy of ram.

If $W > Pe$; the pile is driven into penetrable ground and the value of

$$\eta = (W+Pe^2)/(W+P) \tag{11.7}$$

If $W < Pe$; the pile is driven into penetrable ground and the value of

$$\eta = (W+Pe^2)/(W+P) - ((W-Pe)/(W+P))^2 \tag{11.8}$$

Values of η in relation to e and to the ratio P/W are tabulated in **Table-11.4**

P is the weight of the pile, anvil, helmet and follower (if any) in kN.

Where, the pile finds refusal in rock, the value of P should be substituted as 0.5P in the above expressions for η.

e is the coefficient of restitution of the materials under impact is given in **Table 11.4**.

Where single-acting or drop hammers work in inclined leader guides for raker piles; the percentages given in **Table 11.3** should be deducted from the calculated bearing value.

Table 11.3: Reduced Bearing Values for Raking Piles

Rake	Percent reduction
1 in 12	1.0
1 in 10	1.5
1 in 8	2.0
1 in 6	3.0
1 in 5	4.0
1 in 4	5.5
1 in 3	8.5
1 in 2	14.0

11.7.4.1 Efficiency of Blow

The value of the coefficient of restitution, *e*, has been determined experimentally for different materials and conditions is approximately in listed in **Table 11-4**.

The efficiency of the blow given by the formula can be obtained from the following Table for various combinations of *e* with the ratio P/W, provided that W is greater than P*e* and the piles are driven into penetrable ground. For other cases and if the point of the pile is on rock, the efficiency should be calculated as specified in Hiley Formula.

Table 11.4: Efficiency of Blow

P/W Ratio	*e* = 0.5	*e* = 0.4	*e* = 0.32	*e* = 0.25	*e* = 0
½	0.78	0.72	0.7	0.69	0.67
1	0.63	0.58	0.55	0.53	0.50
$1\frac{1}{2}$	0.55	0.50	0.46	0.44	0.40
2	0.50	0.44	0.40	0.37	0.33
$2\frac{1}{2}$	0.45	0.40	0.36	0.33	0.28
3	0.42	0.36	0.33	0.30	0.25
4	0.36	0.31	0.28	0.25	0.20
5	0.31	0.27	0.25	0.21	0.16
6	0.27	0.24	0.23	0.19	0.14

Table 11.5: Temporary Compressions (mm)

Form of Compr-ession	**Material**	**Easy Driving**	**Medium Driving**	**Hard Driving**	**Very Hard Driving**
Pile head and cap, ***Cc***	Short dolly in helmet or driving cap.*	1.3	2.5	3.8	5.0
	75mm packing under helmet or driving cap.	1.8	3.8	5.6	7.6
Pile length, ***Cp***	Steel pile, steel tube, or steel mandrel for cast-in-place	0.25L	0.50L	0.75L	1.0L
Quake ***Cq***	Ground surrounding pile and under pile point.	1.3	1.3 to 2.5	3.8 to 6.4	1.3 to 3.8

**Note: Pile length L is measured in metres*

- If these devices are used in combination, the compressions should be added together.
- Under typical site conditions, it is normal to select a ratio of drive weight to weight of pile plus cap as 1:2 to 1.5:1. Driving caps, or flat anvil blocks, are necessary to protect the piling heads during driving.
- The characteristics of ground under dynamic induced loads are outlined below, which must be considered during driving of piles.

11.8 Pile Setup and Relaxation

As a pile is driven into the ground, it disturbs and displaces the soil. When the soil that surrounds the pile recovers from this installation disturbance, a time-dependant increase in pile capacity often occurs known as “setup” or “pile freeze”.

On the other hand, a reduction in capacity is called “relaxation”. The dissipation of excess pore pressure during pile driving rules the setup and relaxation.

Setup usually occurs in organic and inorganic saturated clay, loose to medium dense silt, sandy silt, silty sand, and fine sand. Minor or no setup occurs in highly silty low-plasticity cohesive materials. It is observed that in sand and gravel setup may not be a significant factor in long-term pile capacity. In cohesive soils (clay) or a mixture of fine-grained granular and cohesive soils (clayey sand, or clayey silt), the excess pore water pressure induced by pile driving takes longer to dissipate. It has been observed that soft clays setup more than stiff clays (Long *et al.*, 1999). On the other hand, in fine-grained granular soils (fine sand, or non-plastic silt), the excess pore water pressure induced by driving dissipates relatively fast (almost while driving).

11.8.1 Pile Restrike

During driving of pile, the pile setup or relaxation phenomenon may give misleading results about the refusal; therefore, pile re-strike is carried out. It is the action of hitting a pile after the initial driving and waiting a certain amount of time to allow for dissipation of pore water pressure and to evaluate the changes in the capacity of the pile.

There are two types of re-strikes defined depending on the amount of time waited.

a) **Set-Checks**: are when the re-strike is performed within a time not to exceed the following working day after the initial driving of the pile.

b) **Re-drives**: are when the re-strike is performed after one working day following the initial driving of the pile.

c) **Practical Refusal:** Practical refusal is the point at which there is a risk of damaging the pile or equipment with continued driving. This is generally caused by the pile encountering dense material and more specifically; it may be considered as 240 blows per foot (300 mm) penetration.

11.9 Choice of Sheet Piling Sections for Driving

The pile section chosen should be capable of being driven through the various strata to the required penetration depth. Drivability of a piling section is a function of:

a. its cross-sectional properties;

b. length and ;

c. the steel grade used

Shape and cross-sectional properties of sheet pile are important parameters which derive the ground resistance. The greater the surface area of the piling profile, the greater the driving force required. Also, the geometry of the pile section may cause plugging of the pilings in most cohesive soil strata and also in dense, granular strata.

The capacity to transmit the driving force into the ground depends on the area of cross-section of pile and grade of steel being used. For similar section-properties of pile, as the steel grade increases, the stress that the pilings can withstand also increases; thus, logically, the higher-yield steel pilings are more resistant to head or toe deformation than the same section in a lower steel grade. Following table may be used as a guidance to correlate the SPT value of ground (stiffness of ground) with wall modulus (section modulus of pile) to limit the driving stresses.

Table 11.6: Correlation between SPT ‘N’ values of Ground vs. Wall Modulus

SPT ‘N’ value	**Wall modulus (cm^3/m)**	
	Low-yield steel	**High yield steel**
0 - 10	500	-
11 - 20	-	500
21 - 25	1000	-
26 - 30	-	1000
31 - 35	1300	-
36 - 40	-	1300
41 - 45	2300	-
46 - 50	-	2300
51 - 60	3000	-
61 - 70	-	3000
71 - 80	4000	-
80 - 140	-	4000

In addition, for the driving equipment to be acceptable the pile stresses which are stipulated by the wave equation or dynamic formulae shall not exceed allowable values. For steel piles, maximum compressive driving stresses shall not exceed 90% of the minimum yield strength of the pile material.

Example: 11.1

The weight of a 25 m long open ended steel pipe pile including the weight of ram and soil plug is 276 KN. The ultimate bearing capacity of pile is 9500 KN and the rated energy for the set of hammer is as follows;

Weight of Ram	Drop height
i) 350 KN	1.50 M
ii) 200 KN	1.50 M
iii) 140 KN	1.20 M
iv) 80 KN	1.20 M

Evaluate the drivability of pile at termination depth for the above cases of impact hammer.

Given:

The details of pile is given below;

Ultimate bearing capacity of pile	Q	=	9500 KN
Hammer coefficient	K	=	0.90
Total Length of pile	L	=	25.0 M
Area of C/S of pile	A	=	668.4 sq.cm
Total weight of pile + soil plug	P	=	227.0 KN
Coefficient of restitution	e	=	0.50
Modulus of elasticity of steel	E	=	210000 MPa

Evaluate the drivability of pile at termination depth for the following cases of impact hammer.

Solution:

Drivability of pile upto the designed depth is related to the final set of pile or penetration per blow. In accordance with the hammer manufacturer’s recommendations, a penetration of 25mm (1 inch) per 10 blows should be considered the limit for the use of impact hammers. However, a penetration of 10mm per blow or more is highly desirable in general driving. To assess the drivability of pile for above cases, final set in pile is estimated as given below.

Case-i:

Weight of Ram	w	=	350 KN
Ht. of fall for hammer	h	=	1.50 M
Maximum driving resistance (Ultimate bearing capacity of pile)	Q	=	9500 KN
Rated Energy of Hammer, Er = w.h		=	525.0 KN.m
Hammer coefficient	K	=	0.90
Working Energy of Hammer; W=K.Er	W	=	472.5 KN
Total Length of pile	L	=	25.0 M
Area of C/S of pile	A	=	668.4 sq.cm
Total weight of pile + soil plug	P	=	227.0 KN
Coefficient of restitution	e	=	0.50
Modulus of elasticity of steel	E	=	210000 MPa
	P/W	=	0.48
$\eta = \frac{W+Pe^2}{W+P}$	η	=	0.64
	AE	=	14036510 KN
	Cc	=	2.50 mm
Effective compression length of pile (for medium dense strata)		=	0.50 L
Cp = R*(0.5*L) / AE	Cp	=	8.88 mm
Cq	Cq	=	2.00 mm
C = Cc + Cp + Cq	C	=	13.38 mm
Final set of pile; S =[η (W) / Q] - C/2	**S**	**=**	**25.31 mm**

As final set in pile is very high, it is oversized hammer. It may damage the pile while driving.

Case-ii:

Maximum driving resistance (Ultimate bearing capacity of pile)	Q	=	9500 KN
Weight of Ram	w	=	200 KN
Ht. of Fall for Hammer	h	=	1.50 M
Rated Energy of Hammer, Er = w.h		=	300.0 KN.m
Efficiency of Hammer	K	=	0.90
Working Energy of Hammer; W=K.Er	W	=	270.0 KN
Total Length of pile	L	=	25.0 M
Area of C/S of pile	A	=	668.4 sq.cm
Total weight of pile + soil plug	P	=	77.4 KN
Coefficient of restitution	e	=	0.50
E	E	=	9 MPa
	P/W	=	0.29
$\eta = \frac{W+Pe^2}{W+P}$	η	=	0.62
	AE	=	594 KN
Cc	Cc	=	2.50 mm
Effective compression length of pile (for medium dense strata)		=	0.50 L
Cp = R*(0.5*L) / AE	Cp	=	8.88 mm
Cq	Cq	=	2.00 mm
C = Cc + Cp + Cq	C	=	13.38 mm
Final set of pile; S =[η (W) / Q] - C/2	**S**	**=**	**10.81 mm**

Final set in pile (at termination depth) is around 10mm; hammer size can further be optimised.

Case-iii:

Maximum driving resistance (Ultimate bearing capacity of pile)	Q	=	9500 KN
Weight of Ram	w	=	140 KN
Ht. of Fall for Hammer	h	=	1.20 M
Rated Energy of Hammer, Er = w.h		=	168.0 KN.m
Efficiency of Hammer	K	=	0.90
Working Energy of Hammer; W=K.Er	W	=	151.2 KN
Total Length of pile	L	=	25.0 M
Area of C/S of pile	A	=	668.4 sq.cm
Total weight of pile + soil plug	P	=	108.3 KN
Coefficient of restitution	e	=	0.50
E	E	=	9 MPa
	P/W	=	0.72
$\eta = \frac{W+Pe^2}{W+P}$	η	=	0.61
	AE	=	594 KN
Cc	Cc	=	2.50 mm
Effective compression length of pile (for medium dense strata)		=	0.50 L
Cp = R*(0.5*L) / AE	Cp	=	8.88 mm
Cq	Cq	=	2.00 mm
C = Cc + Cp + Cq	C	=	13.38 mm
Final set of pile; S =[η (W) / Q] - C/2	**S**	**=**	**3.05 mm**

This is optimal sized hammer for the case.

Case-iv:

Maximum driving resistance (Ultimate bearing capacity of pile)	Q	=	9500 KN
Weight of Ram	w	=	80 KN
Ht. of Fall for Hammer	h	=	1.20 M
Rated Energy of Hammer, Er = w.h		=	96.0 KN.m
Efficiency of Hammer	K	=	0.90
Working Energy of Hammer; W=K.Er	W	=	86.4 KN
Total Length of pile	L	=	25.0 M
Area of C/S of pile	A	=	668.4 sq.cm
Total weight of pile + soil plug	P	=	124.8 KN
Coefficient of restitution	e	=	0.50
E	E	=	9 MPa
	P/W	=	1.44
$\eta = \frac{W+Pe^2}{W+P}$	η	=	0.71
	AE	=	594 KN
Cc	Cc	=	2.50 mm
Effective compression length of pile (for medium dense strata)		=	0.50 L
Cp = R*(0.5*L) / AE	Cp	=	8.88 mm
Cq	Cq	=	2.00 mm
C = Cc + Cp + Cq	C	=	13.38 mm
Final set of pile; S =[η (W) / Q] - C/2	**S**	**=**	**-0.26 mm**

This is undersized hammer and pile cannot be driven upto the designed depth.

Example: 11.2

An open-ended pipe pile is to be driven through stratified soil medium upto 28m depth. The details of pile, hammer and driving medium are given below;

Details of Sub-Soil Strata

E.L. of Soil Strata			Type of Soil	Depth (m)	SPT 'N'$_{60}$	γ'	c-value	δ	Nq	τ_{max}	q_{max}
From E.L	to	E.L				(KN/m^3)	(KN/m^2)	degrees		(Kpa)	(Kpa)
-6.0 m	to	-10.0 m	Medium Sand	4.0	21.0	8.0	0.0	25.0	20.0	81.3	4.8
-10.0 m	to	-16.5 m	Medium Sand	6.5	25.0	9.0	0.0	25.0	20.0	81.3	4.8
-16.5 m	to	-18.0 m	Silty Clay	1.5	7.0	9.0	40.0	0.0	1.0	-	-
-18.0 m	to	-28.0 m	Medium Sand	10.0	27.0	9.6	0.0	25.0	20.0	81.3	4.8
-28.0 m	to	-36.0 m	Dense Sand	8.0	35.0	10.0	0.0	30.0	30.0	95.7	9.6

Top elevation of Ground = - 6.00 M Lvl.
Founding level of Pile = - 34.00 M Lvl.
Ground water table = (+) 1.0 M
Unit weight of water = 10.00 KN/m^3

Details of pile and hammer

Outer dia of pile (Do) = 0.916 m
Inner dia of pile (di) = 0.900 m
Area of annular base (An) = 0.023 m^2
Area of plug base (Ab) = 0.636 m^2
Yield stress of steel (fy) = 420 MPa
Coeff. of restitution of steel (e) = 0.5
Weight of Ram (w) = 220 KN
Ht. of Fall for Hammer (h) = 1.20 m
Rated Energy of Hammer (Er) = 264 KN.m
Efficiency of Hammer (K) = 0.90
Energy of Hammer (W=K.Er) = 237.6 KN.m

Evaluate the drivability of pile upto the founding strata (-34.0M level).

Solution:

To assess the drivability of pile, driving resistance caused by the soil, corresponding pile sets and driving stresses in pile are estimated for each strata and presented in tabular form. The details are on the basis of solution presented in Example - 11.1.

Assessment for the driveability of pile

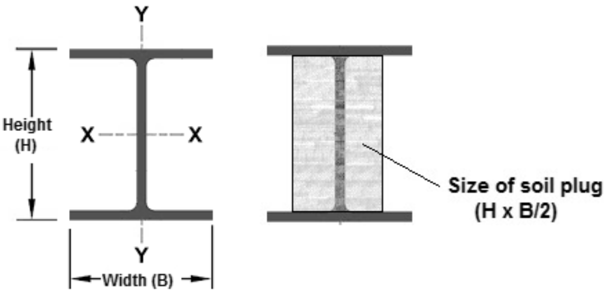
Soil Strata	LEVEL	Thickness of Layer (M)	SPT 'N'$_{60}$	Type of driving	Comp. Length	γ	Average Pr. (p_0)	Su	Driving Resistance (Q)	Wt of soil Plug	Wt of soil Pile	Wt of soil + Pile (P)	P/W	$\eta = \frac{W+Pe^2}{W+P}$	Cp	Cc	Cq	C = Cp+Cc+Cq	Set in Pile (S)	Driving Stress in pile	Remarks
						Kpa	Kpa	Kpa	(KN)	(KN)	(KN)	(KN)			mm	mm	mm	mm	mm		
Layer-1	-6.00 -10.0	4	12	Easy	0.25 L	8.0	16.0	0.0	186.4	20.4	50.2	70.5	0.3	0.8	0.0	1.3	1.3	2.6	1054.7	0.02 fy (O.K.)	Drivable
Layer-2	-10.0 -16.5	6.5	25	Medium	0.50 L	9.0	61.3	0.0	1281.5	57.6	50.2	107.7	0.5	0.8	1.5	2.5	2.0	6.0	139.0	0.13 fy (O.K.)	Drivable
Layer-3	-16.5 -18.0	1.5	7	Easy	0.25 L	9.0	97.3	40.0	1060.5	66.2	50.2	116.3	0.5	0.8	0.7	1.3	1.3	3.3	167.2	0.11 fy (O.K.)	Drivable
Layer-4	-18.0 -24.4	6.35	27	Medium	0.50 L	9.5	134.2	0.0	3870.9	104.5	50.2	154.7	0.7	0.7	7.8	2.5	2.0	12.3	37.1	0.40 fy (O.K.)	Drivable
Layer-5	-24.4 -28.0	3.65	27	Medium	0.50 L	9.5	164.3	0.0	4720.8	126.6	50.2	176.8	0.7	0.7	11.4	2.5	2.0	15.9	26.3	0.49 fy (O.K.)	Drivable
Layer-6	-28.0 -30.0	2	40	Hard	0.75 L	10.0	164.3	0.0	6114.2	139.3	50.2	189.5	0.8	0.7	24.1	3.8	5.0	32.9	9.5	0.64 fy (O.K.)	Drivable
Layer-6	-30.0 -32.0	2	40	Hard	0.75 L	10.0	164.3	0.0	6695.3	152.0	50.2	202.2	0.9	0.7	28.6	3.8	5.0	37.4	4.5	0.70 fy (O.K.)	Near Refusal
Layer-6	-32.0 -34.0	2	40	Hard	0.75 L	10.0	164.3	0.0	7141.1	164.8	50.2	214.9	0.9	0.6	32.9	3.8	5.0	41.7	0.6	0.75 fy (O.K.)	Refusal

Example: 11.3

Using the soil strata as given in example: 11.2, assess the driveability of steel H-beam upto the 28m depth. The details of pile and pile driver are given below;

Plan Area of H-Beam H = 1.05 m
B = 0.46 m
Area of annular base (An) = 0.028 m^2
Yield stress of steel (fy) = 355.0 MPa
Weight of Ram (w) = 100.0 KN
Ht. of Fall for Hammer (h) = 1.20 m
Efficiency of Hammer (K) = 0.9

Solution:



Normal H-Pile H-Pile with plugged base area

Area of plug base (H*B/2) (Ab) = 0.166 m^2
Yield stress of steel (fy) = 355.0 MPa
Coeff. of restitution of steel (e) = 0.5
Weight of Ram (w) = 100.0 KN
Ht. of Fall for Hammer (h) = 1.20 m
Rated Energy of Hammer (w.h) (Er) = 120.0 KN.m
Efficiency of Hammer (K) = 0.9
Energy of Hammer (W=K.Er.) = 108.0 KN.m

Assessment for the driveability of H-Beam

Soil Strata	LEVEL	Thickness of Layer (M)	SPT 'N'$_{60}$	Type of driving Comp. Length	γ	Average Pr. (p_0)	Su	Driving Resistance (Q)	Wt of soil Plug	Wt of soil Pile	Wt of soil + Pile (P)	P/W	$\eta = \frac{W+Pe^2}{W+P}$	Cp	Cc	Cq	C = Cp+Cc+Cq	Pile setup (S)	Driving Stress in pile	Remarks
					Kpa	Kpa	Kpa	(KN)	(KN)	(KN)	(KN)			mm	mm	mm	mm	mm		
Layer-1	-6.0 m -10.0 m	4.00	12	Easy 0.25 L	8.0	16.0	0.0	117.1	2.7	61.2	63.9	0.59	0.72	0.0	1.3	1.3	2.6	664.2	0.01 fy (O.K.)	Drivable
Layer-2	-10.0 m -16.5 m	6.50	25	Medium 0.50 L	9.0	61.3	0.0	696.2	7.5	61.2	68.7	0.64	0.71	0.7	2.5	2.0	5.2	107.3	0.07 fy (O.K.)	Drivable
Layer-3	-16.5 m -18.0 m	1.50	7	Easy 0.25 L	9.0	97.3	40.0	678.7	8.6	61.2	69.8	0.65	0.71	0.4	1.3	1.3	3.0	110.8	0.07 fy (O.K.)	Drivable
Layer-4	-18.0 m -24.4 m	6.35	27	Medium 0.50 L	9.5	134.2	0.0	1912.3	13.7	61.2	74.8	0.69	0.69	3.2	2.5	2.0	7.7	35.3	0.19 fy (O.K.)	Drivable
Layer-5	-24.4 m -28.0 m	3.65	27	Medium 0.50 L	9.5	164.3	0.0	2464.0	16.5	61.2	77.7	0.72	0.69	4.9	2.5	2.0	9.4	25.4	0.25 fy (O.K.)	Drivable
Layer-6	-28.0 m -30.0 m	2.00	40	Hard 0.75 L	10.0	164.3	0.0	2997.7	18.2	61.2	79.4	0.74	0.68	9.7	3.8	5.0	18.5	15.3	0.30 fy (O.K.)	Drivable
Layer-6	-30.0 m -32.0 m	2.00	40	Hard 0.75 L	10.0	164.3	0.0	3371.9	19.9	61.2	81.1	0.75	0.68	11.8	3.8	5.0	20.6	11.4	0.34 fy (O.K.)	Drivable
Layer-6	-32.0 m -34.0 m	2.00	40	Hard 0.75 L	10.0	164.3	0.0	3746.2	21.5	61.2	82.7	0.77	0.67	14.1	3.8	5.0	22.9	8.0	0.38 fy (O.K.)	Near Refusal

11.9.1 Vibratory Pile Drivers

Vibratory pile drivers apply vibrations to the pilings in order to enable them to penetrate certain soil strata. The principle of vibratory driving is the reduction of skin friction between the pile and the soil.

The vibrations will temporarily disturb the soil around the pile, causing minor liquefaction, which results in a noticeable decrease in resistance between soil and pile. This enables the pile to be driven into the ground with very little added load (i.e., its own weight plus the weight of the driver). The vibratory driver generates oscillations inside a vibration case in which eccentric weights are gear-driven by one or more motors.

The soils best suited to vibration work are non-cohesive soils, i.e. gravel or sand, provided that the soils are not too dense. In cases where SPT (Standard Penetration Test) N-values of over 50 are present, vibro-driving will be difficult. Pre-augering prior to vibro-driving the piles and water jetting whilst vibro-driving can be utilised to loosen the dense soil strata. Vibratory hammers operating at higher amplitudes are normally effective in more difficult soils.

Clay soils have a dampening effect and reduce the effectiveness of the energy transferred to the pile. Vibratory driving is therefore difficult where firm or stiff clay soils are encountered. High amplitude hammers are likely to yield the best results in such conditions. *Should the vibratory hammer reach a refusal rate of less than 75mm of pile penetration per minute, Impact Driving can be used to progress the piles further*.

The vibratory pile driver is also a very efficient piling extractor. The decrease in resistance between soil and pile enables extraction to occur with a greatly reduced upward pull, compared with the static extraction force that would be required to extract a similar pile. The frequency of the standard range of vibratory drivers varies from 800-1800rpm and the centrifugal forces go up to 5000kN. The recently introduced higher frequency drivers give a range up to 3000rpm. The resulting high vibrations attenuate very rapidly, thus causing very few problems to adjacent properties. The penetration performance depends mainly on the soil conditions.

The most appropriate soils for vibro-driving are non-cohesive soil, gravel or sand, especially when they are water-saturated. With mixed or cohesive soils, vibro-drivers can be used where there is a high-water content. Artificially dewatered sands, on the other hand, can be very resistant to vibratory driving.

11.9.2 Advantages of Vibratory Driving Solutions

Vibratory driving provides;

- Fast and economical solution suitable to loose or cohesionless soil conditions
- Capable of reducing noise and vibration output
- Single or double hammer size options available, adaptable to suit the size of rig and/or any site restrictions
- Adaptable for use with small excavators
- Can be efficiently used to extract piles
- Digital reporting to monitor pile installation

As a guide to determining the size of a vibratory driver, the following formula may be used:

a) Force

$$F = 15\left(t + {}^{2G}/_{100}\right) \text{ [KN]} \qquad (11.12)$$

Where, F = centrifugal force,

t = driving depth in m,

G = mass of the pile in kg.

It is generally recognized that a penetration rate of approximately 10cm (4 inch) per minute be used as a limit. This is to be considered only as an indication for the control of possible vibration nuisance.

b) Amplitude

$$A = \ (\ 2 \text{ x } M_t \ / \ M_v\ ) \qquad (11.13)$$

M_t = Eccentric Moment in Kg.m

M_v = Total Vibrating Weight

$\mathrm{M_v}$ = Weight of pile + Weight of hammer in KN

A = Amplitude in mm.

The vibrating weight ($\mathrm{M_v}$) is the sum of all the weights of the vibrating mass = Dynamic weight (vibrating mass) + Pile weight + Clamping device including all plates or clamps

For vibratory driving assessment, theoretical working amplitude 'A' should be targeted as more than 3mm.

c) Acceleration

The acceleration of the driving material acts on the grain structure of the encountered soil. The grain structure stratification should be constantly moved during vibration in order to approach the "pseudo-liquid" state in the ideal case. The product of the "working amplitude" and the square of the angular velocity yield the acceleration of the driving material.

$$\mathrm{a} = \mathrm{A.}\,\omega^2 \quad [\mathrm{m/s^2}] \tag{11.14}$$

$$\omega = 2.\pi.n/60$$

Where, ω is the angular velocity in radians /sec. and "n" in rpm.

Based on experience, the driving acceleration "a" should be ≥ 100 $\mathrm{m/s^2}$.

d) Bearing Capacity of Vibro-driven piles

The bearing capacity of vibro-driven piles can be predicted using Snip formula. This empirical formula is used in Soviet practice and is predicated on observed behaviour of full-sized piles in the Soviet Union:

$$Q_t = \lambda\left(\frac{25.5\,P}{A_o\,f} + W_t\right) \tag{11.15}$$

Where,

Q_t = ultimate, static, compressional bearing capacity of pile, in kN;

P = power used by vibrator to drive pile, in kilowatts;

A_o = displacement amplitude of vibrator, in cm;

f = frequency of vibrator, in Hz;

W_t = total weight (force) of vibrator and pile, in kN; and

λ = soil coefficient reflecting the influence of driving on soil properties (e.g., in Soviet practice λ is provided in Table 11.7).

Table 11.7: Soil Coefficient for Snip/Soviet Formula

Soil Type	**Soil Coefficient; λ**
Saturated Sand	4.0 - 7.5
Moist Sand	2.5 - 4.0
Dry Sand	2.5 - 4.0
Sandy Clay	2.5 - 5.0
Silty Clay	2.2 - 4.5
Clay	2.0 - 4.5

e) Choice of section

Once the static calculation is done and section modulus, pile length as well as steel grade are defined, it has to be checked, whether length of pile and section modulus are adequate for installation in the given soil conditions.

Example: 11.4

An open ended steel pile is to be driven through medium dense strata using vibratory pile driver. The details of vibratory pile driver and pile are as given below;

- ***Vibratory Pile Driver***

Eccentric Moment	-	*50 Kg.m*
Hydraulic Power	-	*300 KW*
Maxm. Frequency	-	*1500 r.p.m (25 Hz)*
Maxm. amplitude	-	*32 mm*
Maxm. Centrifugal force	-	*1258 KN*
Total wt. of pile driver	-	*5720 Kg*

- ***Pile***

Wt. of pile	-	*160 Kg/m*
Total length of pile	-	*18.0 m*

The soil stratum is in saturated condition. Evaluate the effectiveness of above pile driver by estimating the followings;

a) Required centrifugal force to drive the pile
b) amplitude and acceleration of pile driving system
c) ultimate load bearing capacity of pile

Solution:

Eccentric Moment Mt = 50.00 Kg.m
= 0.500 KN.m
Total wt. pile = 2880 Kg
Wt. of hammer = 5720 Kg
Total wt. of driving unit Mv = 8600 Kg
= 86.0 KN

a) Centrifugal force required for Hammer (Fv)

Fv = 15 x (t + 2G/100) KN

t = 18.0 m
G = 2880 Kg
Fv = 1134 KN

b) Amplitude of Driving Unit (A)

A = 2. Mt / Mv

Mt = 0.50 KN.m
Mv = 86 KN
A = 0.0116 m
= 11.6 mm

Acceleration of Driving Unit (a)

$a = A.\ \omega^2$

A = 11.63 mm
= 0.0116 m

$\omega = 2.\ \pi\ .n\ /60$

ω = 157.00 rad./sec
a = 286.91 m/sec^2
> 100 m/sec^2
(O.K)

c) Ultimate Bearing Capacity of Pile (Q_t)

The bearing capacity of vibro-driven piles can be predicted using Snip formula as used in Soviet practice;

$$Q_t = \lambda\left(\frac{25.5\,P}{A_o\,f} + W_t\right)$$

Where,

Q_t = ultimate bearing capacity of pile, in kN;

P = power used by vibrator = 300 KW

A_o = displacement amplitude = 1.16 cm

f = frequency of vibrator = 25 Hz

W_t = total weight of vibrator and pile = 86.0 kN

λ = 5.0 (snip coefficient for medium dense saturated sand)

$$\mathrm{Q_t} = 5.0\,\mathrm{x}\left(\frac{25.5\,\mathrm{x}\,300}{1.16\,\mathrm{x}\;25} + 86\right)$$

= **1749 KN**

11.10 Installation Procedure of Sheet Pile

Setting and driving techniques of sheet piles may vary according to the site conditions and/or the contractor's level of experience, however, Sheet piling can be installed using one of three driving methods as outlined below;

11.10.1 Pitch and Drive Method

Pitch and drive is the simplest method for sheet pile driving, it involves driving each sheet pile to full depth before pitching the next one as depicted in **Figure 11.4**. However, it is only suitable for loose soils and short piles. With the pitch and drive method, a guide is not installed. The sheet piles are directly threaded and driven to the ground. But each sheet pile has to be installed sequentially. In this method, the sheet piles are interlocked one at a time which may cause to the leaning of some piles or some may rotate. Therefore, the pitch and drive method is used for temporary walls or when the sheet piles are short.

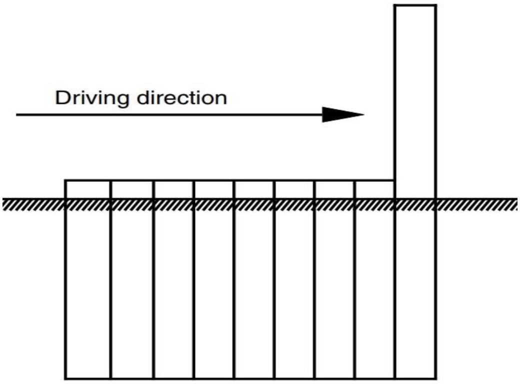


Figure 11.4 Schematic Arrangements of Pitch and Drive Method

11.10.2 Panel Driving Method

The purpose of panel driving is to ensure that the sheet piles are driven with good alignment and verticality. The risks of driving difficulties and problems of de-clutching also can be minimised. The general procedure of panel driving is outlined below;

i) Installation of Template

Panel driving involves a template which acts as a guide-structure to ensure that piles will be placed and driven to the correct alignment. The purpose of the template is to properly align the sheets during the setting or pitching and to keep the sheets in alignment during the driving. Since sheet piles are heavier steel elements, the templates should be very sturdy.

ii) Set the Panels of Piling

The template is first inserted into the ground to serve as a guide for the incoming sheet piles. After placing the guide, each sheet pile is threaded vertically and put in position. In this step, all the sheet piles are interlocked together, which, ensures that the sheets will not lean and stand. In this process, to thread the sheet piles a long crane is required that can reach high elevations for lifting the sheet piles. For longer sheet piles, one may need a crane twice the size of the pile.

iii) Drive the panels of Sheet

After pitching the piles in correct alignment, the driving machine drives each pile into the ground into segments. As a whole panel of piles is pitched there is no need to drive all piles fully to maintain piling operations. Piles driven full length in one operation are more prone to deflect and go off line. This is particularly true when the soil contains debris, boulders, or other obstructions. Subsequent piles are guided by the deflected section; and within a short length of wall, pile driving comes to a halt. The sheeting must then be pulled, and the wall has to be restarted. To minimize this problem, first set a panel of sheets and then drive the sheets in increments using a defined sequence as shown in **Figure-11.5**, below.

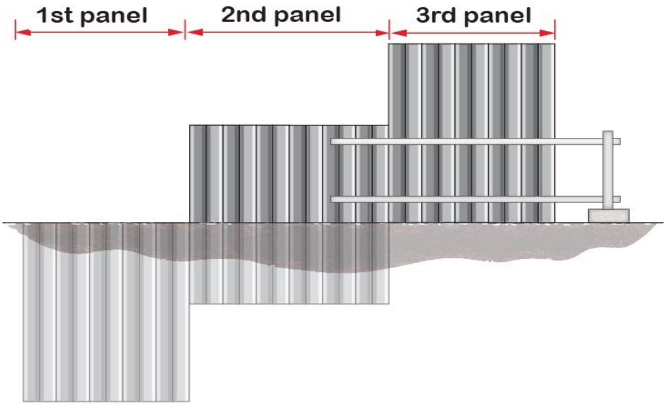


Figure 11.5 Schematic Arrangement of Panel Driving System

The magnitude of the increment will be determined by the soil conditions. In general, the harder the driving, the less the driving increment; usually 2 meters in easy driving and 1 meter or less in case of denser soil. Panel driving allows the piles to be guided by previously driven piles, and it lessens the possibility of driving the piles out of interlock.

11.10.3 Staggered Driving

(Dense Sand, Gravel or Rock)

In difficult soil conditions such as dense sand, gravel or rock, panel installation combined with staggered driving is more suitable in driving process. The piles are installed between guide frames and then driven in short steps as follows: piles 1, 5 and 3 first; then piles 2 and 4 as schematically shown in **Figure-11.6**. If the soil is very dense sand, gravel or rock, piles 1, 5 and 3 can be reinforced at the toe. In this case, these piles are always driven first and piles 2 and 4 in the second stage.

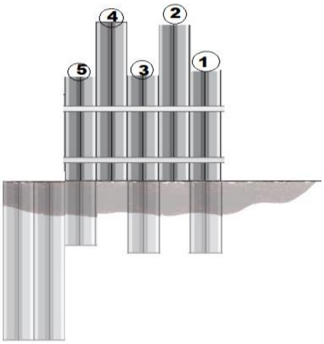


Figure 11.6 Schematic Arrangement of Staggered Driving System

11.11 Installation of Combi-Sheet Pile Wall

Combined walls are piling walls that comprise high modulus structural components interspaced by lighter sheet pilings. The high modulus components – known as king pilings – can be tubular, box, bearing or other types of fabricated pilings. It is essential that a stable, heavy, adequately rigid and straight pile-driving template frame, adapted to suit the length and weight of the pilings, be provided.

Driving of the king pilings must be carried out with extreme care in order to ensure that they are embedded straight and vertical, or at a prescribed batter, thereby guaranteeing that they are parallel to each other and at the required spacing. After successful completion of king pile driving, the intermediate light piling sections can be set and driven.

The installation of the combined wall requires detailed attention to the equipment, templates and the manner of installation which are discussed below in detail;

11.11.1 Installation Process

The installation of the combined wall requires detailed attention to the equipment and the manner of installation. When installing a combined wall (with tubular or H-beams as king piles), the king piles should always be placed first by using a driving template because these are the stiffest elements. The more flexible intermediate sheet piles should be installed afterwards as shown in **Figure-11.7** below.

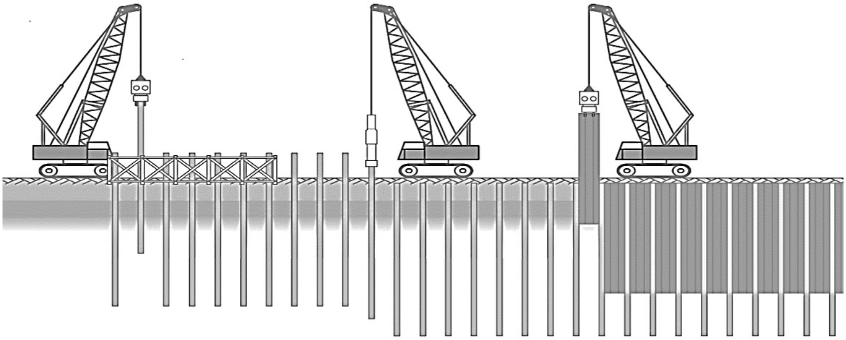


Figure 11.7 Order of Installation of Combined Wall

It is advised to use a crane with a leader or to utilize a two-level driving template. The supplementary equipment required depends on whether the installation will be done from water or land. For installing the combined wall from water either a pontoon or a jacket structure is needed for the crane. An additional pontoon or barge can be attached to the main structure for the supply of tubes or king piles and intermediate sheet piles.

11.11.1.1 The Order of Installation

While driving of new pile in ground, it causes compaction and lateral deformation in surrounding soil, therefore, the installation order of pile is very important to avoid the cumulative displacement in soil. Right order of installation of piles ensures the best possible results regarding the horizontal positioning of the tubes and their verticality as are discussed

i) Installation of King Piles

a) Before pitching and driving heavy tubular piles, fixing of a sturdy template is very important. In order to fix the position of the tubes in the horizontal direction a driving template is required. Templates are usually constructed with beams and/or hollow sections. The short cross beams are placed in such a way that the right distance between the tubes can be assured. Welded onto the cross beams, are small “notches” to hold the interlock in place and to prevent the rotation of the tubes while driving. As a tolerance, the space between the interlocks and the notches should not be more than a few millimetres. A typical configuration of a driving template should always have an odd number of spaces for the tubes as shown in the figure below.

b) After pitching all king piles in a panel or template, first and last pile should be driven in priority, and then alternate pile shall be followed. The order of installation is important to guarantee the best possible results regarding the horizontal positioning of the tubes and their verticality. The sequence of installation is depicted by the numbers in the figure below, i.e., driving may have to proceed from outside to inside. The reason for installing the piles in this order is geotechnical. Driving of tubular piles in firm strata cause compaction of soil during vibration and also have plugging effect. If pile will be driven serially, from left to right, the sub-soil strata will be compacted in this order and due to differential compaction of soil on either side of pile, it can lead to non-verticality to the adjacent installed tube. Based on site experience, the order of installation are generally advised as shown in **Figure11.8(a)**.

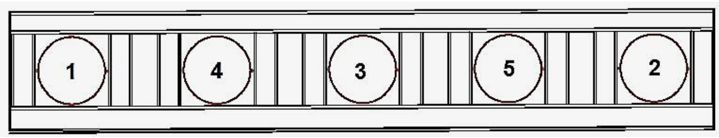


Figure 11.8(a) Sequence of King Pile Driving

c) If templates can accommodate five (5) nos. of piles, first and last piles are driven first, it will provide confinement to the remaining piles of panel and differential compaction as well as lateral shift in pile during driving get minimised. After driving the outer piles, middle one is driven and remaining 2nd and 4th piles in third sequence. This is an essential requirement for driving of piles in dense sand and hard clayey soil.

d) After the installation of all five piles the template can be moved to the last pile on the right. This pile will act as the first pile for the following section.

ii) Installation of Intermediate sheet piles

Pitching and driving of intermediate sheet piles should always take place after installation of the king piles. The installed king piles act as a guide for infill sheet piles. Also, the intermediate sheet piles are more flexible than the king piles, they can follow small imperfections in the verticality or orientations of piles and get accommodated with main piles while remaining clutched, as shown in **Figure-11.8(b)**, below.

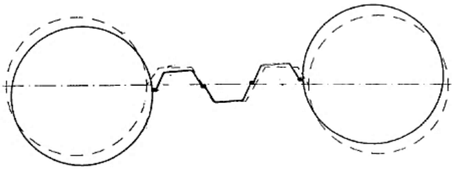


Figure 11.8(b) Driving Tolerances in Combi-wall

iii) Driving Tolerances

Depending on soil conditions, length, shape and size of the intermediate sheet piles, the tolerances on the position of the pile top and verticality are usually very strict. As a general practice, it is recommended that the verticality stays within 10mm/m length of the tube. With regards to the horizontal tolerances, the tubes need to be within 10 to 15mm of the centre line of the tubular combined wall. During the setting and driving operations of the king pilings, a constant check (using theodolites) should be made of their alignment in relation to the wall.

11.11.1.2 Driving Equipment and Mechanism

Current driving technologies allow the use of impact or vibratory hammers to drive tubes and intermediate sheet piles. Use of a vibratory hammer for installation can reduce the risk of damage. In hard soil conditions it is not always possible to install the tubes and sheet piles to the required depth using vibratory methods, so an impact hammer may be necessary. A combination of the two techniques can be judiciously used. Therefore, the driving process of a combined sheet piling wall system is typically a two- or three-step process:

(a) 1. Use a vibro hammer to drive the king piles as far as possible.

(b) 2. Further, use an impact hammer to drive the piles up to the designed toe level (if necessary).

(c) 3. Set the intermediate sheet piling (with attached connector) between the king piles and drive it preferably with vibro-hammer and use the impact hammer at last, if possible.

This combination of driving methods is also used when a plugging effect is required to increase the tip resistance of the tubes. Below mud line, the embedment depth of the intermediate piles for high harbour walls in good bearing soil should be at least 2.5 m; for low walls with slight water pressure difference, it should be at least 1.5 m.

Vibratory hammers should be fitted with adequate clamps to ensure a correct load transfer to the pile. For intermediate double-Z or U sheet piles double clamps are preferred. It is advisable to choose a vibratory hammer with sufficient weight and power to allow good driving speed and penetration while preventing damage to the interlocks caused by overheating.

11.11.1.3 Additional Requirements

(Bracing at the Top of Steel Pipe Piles)

When driving pipe piles, there is a risk of the tops bulging in the upper section of the pipe, particularly in the case of pipes with relatively small wall thicknesses. This can mean that the pipes cannot be driven effectively to the planned depth. In order to prevent bulging, the pile head must be braced. Various methods are available for this; the cross bracing by welding the steel plates of approx. 0.80 m in depth in the head of the pipe pile to form a cross as shown in **Figure 11.9** is very effective.

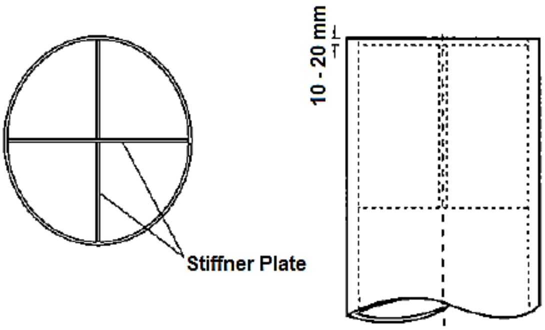


Figure 11.9 Bracing of Pipe Pile at Top

11.11.2 Management in Difficult Driving Conditions

If the driving shows no progress or can only be achieved through excessive driving energy, a possible solution is to apply driving aids, which may consist of followings;

i) Driving Shoe

In hard driving conditions other than rocky soils, the tip of the piles can be reinforced using driving shoes. The design of these driving shoes varies greatly depending mainly on the type of soil, dimensions of the tubes and the driving equipment. Often a driving shoe consists of ring welded on the inside of the tube. In clayey soils the ring and the end of the tube will be bevelled in order to facilitate penetration of the clay. These driving shoes can also be applied to reduce cohesion of the clay on the inside of the tube.

ii) Predrilling

Loosening of hard cohesive and non-cohesive soil strata can be achieved by drilling holes in the ground with an auger. This will reduce point resistance and shaft friction during installation. Less energy is needed to install the piles into the ground, leading to less vibration.

iii) Water-Jetting

With water jetting low water discharges are released under high pressure (10-20 bar) underneath the toe of the pile. This will reduce the toe resistance of the pile in non-cohesive soils. The ascending water lubricates the shaft, reducing the shaft friction. This technique is not to be confused with flushing, which uses high amounts of water with low pressures.

iv) Strengthening the Pile for Hard Driving Conditions

The sites, where hard driving is expected due to presence of hard clay strata a toe adjustment may also be provided in H-beams as indicated below. In case of presence of boulders or rock, a rock shoe may be attached at base of pile.

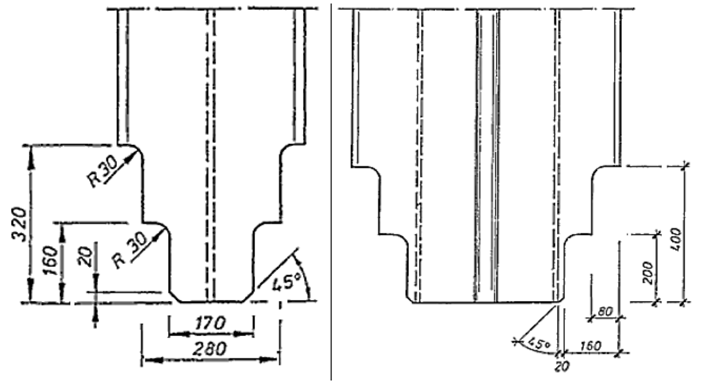


Figure 11.10 Toe Adjustments in H-beams for Driving in Hard strata

Other Practical Considerations

i) Before driving a hot-rolled sheet pile, head of a bolt should be tack welded over the clutch opening on the leading edge of the sheet at the base to help stop soil getting jammed in the clutch while driving. This will keep it clean for threading of the next pile to be driven.

ii) Drive the first two sheets carefully, checking for vertical, longitudinal and lateral alignment. Special attention and time with the initial set-up process are generally required to maintain the alignment of wall and progress of work.

iii) Make sure the boom of the crane is 'luffed' correctly, directly over the top of the line of the piles to avoid bending the pile you are driving. The crane should have enough height to clutch the next pile when the previous pile is still mostly in the air. As a rule of thumb, it should be twice the length of the piles being installed.

iv) Lifting and lowering the pile multiple times as the penetration becomes slower usually helps. Don’t release all the hammer weight when driving, but rather let the hammer cut/work its way through the ground at a steady pace (faster is not always better).

v) If an obstruction is found, keep driving the successive piles until past the obstruction. Later on come back and try to drive the obstructed piles again.

Chapter 12

Creative Applications of sheet Piles in Waterfront Structures

Creativity in engineering is inventing, experimenting, growing, taking risks, breaking rules, and having fun; it is a blend of Logic and Imagination

- Anonymous

12.1 Introduction

A port is a maritime facility which may comprise of one or more wharfs, quays, piers or jetties where ships may dock to load or unload the cargo and passengers. Over the past few decades, the rapid expansion of trade has led to a tremendous increase in cargo handling between the various continents. This continuous growth has motivated to increase the ships sizes which may lead to the need to develop many new marine terminals or upgrade the existing terminals. The port infrastructure has gone through multiple transformations and different types of ports in the world have changed from their original shapes and dimensions to their present configurations. With the rise of the construction of both small and large port-based infrastructures, the need for construction materials has also risen. Among the various available construction materials, sheet piles are one of the key players in construction industry due to its versatile nature and its adaptability for waterfront development. Steel sheet piles are popular pick for retaining walls and waterfront structures and also have number of good reasons as summarised below;

i. Its application is very wide in port infrastructures due to its ability to resist soil and water pressure.

ii. The ability sheet pile to be driven through water makes it the obvious choice for water front structures.

iii. These are tried and tested material to construct bulkhead and quay walls speedily and economically. It can be designed to cater vertical loads and large bending moments.

iv. These are pre-manufactured product with known properties, which can be shipped to a project site and assembled, under diversified conditions, into a permanent wall.

v. Sheet piling can be used in a number of ways in the design and construction of a port operating facility. A sheet pile bulkhead provides a vertical wall, behind which useable land area are created for loading and unloading facility of wharf.

vi. Special high modulus sheet pile wall systems have become a factor in deep water marine facilities construction. This method utilizes large "primary piles" as the main strength units of the wall and secondary units as diaphragms.

Its application is very wide in port infrastructures, but details discussed herein are limited to the use of sheet piles for;

a) for strengthening and deepening of quay wall structures, and;

b) development of bulkhead in various geographical conditions;

12.2 Need for Upgradation of wharves

With increase in ship size, port has to adapt the upcoming changes by increasing the draft of berthing basin. Conventionally to increase the draft, there is trend to move the mooring arrangements towards the sea, where the draughts are deeper. More often, the functionality of those quay structures outdates in less time than they are designed for. In such cases, dismantling the existing old infrastructure and re-building the new one to accommodate the deep draft basin is a very costly effort and not an environment friendly option. Then the question arises, how to deal with these changes. The complete demolition of an existing quay wall construction and replacing by a new structure with a larger depth or draft is often not possible due to the high costs and environmental boundaries.

Then upgrading and deepening of the existing quay walls are the next option. This means, the soil retention height in existing quay walls will be more than they actually designed for. In addition to that, it may entails higher operating loads (mooring, berthing, and cranes) and sometimes storage loads on the structures due to the need to cater the larger ships. Therefore, constructive adjustments or modifications have to be made to the quay walls, to provide sufficient strength and stability. In such modification cases, sheet piles are preferred due to their fast construction, comparative light weight, high resistance to driving stresses, and their suitability of construction above and below the water table. In port construction industry, sheet piles have proven track record.

12.2.1 History of Sheet Pile in Port Construction

With the advancement of time, steel manufacturing had made enough progress that hot-rolled I, Z and U profiles could be manufactured. In 1914, first steel sheet pile element with hot rolled interlocks was manufactured. In this way sheet pile elements with good installation capacity, soil tightness and reduced weight were produced. In the port of Rotterdam, the first steel sheet pile wall was installed in 1927, thus 13 years later the invention of the versatile sheet pile with Larssen interlocks. After the Second World War the combined systems of steel piles and sheet piles were introduced. This system has been in used in the Port of Rotterdam since 1985 and these are one of the key elements for deep draft wharf construction. The most customary wall shapes and wall elements for combined wall are described in next section.

12.2.2 The comparative study of combi-wall systems

For deep draft facility or modification of existing wharf, use of combi-sheet piling with larger section moduli often provide a promising and economical alternative. The combi-sheet pile consists of bearing elements to resist the forces in both vertical and horizontal directions and of intermediate elements that transfer the horizontal loads to the bearing elements. It is constructed by the alternate placing of different types of sections. Long and heavy sections designated as bearing piles alternate with shorter and lighter intermediate piles.

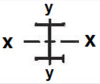
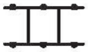
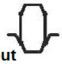
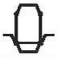
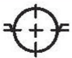
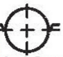
Type of Pile	Rigidity		Torsion	Buckling	Dimensional Stability	Installat-ion	Costs
	Ix	Iy					
	3	1	1	3	4	3	2
	5	3	3	4	4	4	1
 Without Reinforcement Rib	4	3	3	2	3	3	3
 With Reinforcement Rib	5	3	3	3	4	3	3
 Welded Along Seam	5	5	5	5	5	5	4
 Welded With Spiral Seam	5	5	5	5	4	4	5

Figure 12.1 Evaluation of the types of combi-wall system

In order to prevent loss of soil, the connection between these elements must be secured. The problem of combi-wall connections was investigated by [de Gijt and Heijndijk 1996]. In this study the three components pile, sheet wall element and the interlock connection were investigated.

The piles were analysed with regard to bending stiffness in directions, deformation, capacity, dimensional stability, ultimate strength, installation aspects and cost scored between 1(poor) to 5(excellent). The result of the evaluation is presented in **Figure 12.1**. Based on evaluation, preference is given to steel pipe piles. However, in some cases H beams with Z-shaped piles are preferred due to complexity of site. In addition to that, structurally large diameter open-ended pipe piles possess following advantages;

i) It allows a significant unsupported length in the foundation of marine structures as it possesses high column buckling strength. For marine structures like jetties, offshore platforms, piles are well above seabed level and therefore the column buckling effect is significant when compared with other structures. Therefore, it is advantageous to use as pile sections which have relatively high buckling strength.

ii) Tubular piles display high energy absorbing capability. For marine structures like dolphins and fenders, where substantial amount of berthing energy is required to be absorbed, these piles sections are preferred.

iii) It can safely resist large axial loads and provides a good choice for deep foundation when relatively loose or soft sub-soil strata extends to a significant depth.

iv) Expedite and reduce the complications of marine constructions as pile delivery, handling, installation and other complexity are less. The circular pipe piles are also advantageous in active marine environment to minimise the effect of drag and oscillation from waves and currents.

Use of Combi-wall with infill sheet piles of limited height (placing the top level of sheet piles below the tidal zone) as shown in **Figure 12.2** creates a porous wall that nullify the hydrostatic pressure due to tidal variations while fulfilling the other structural requirements. It also cause to reduction in retention heights and effective lateral earth pressures.

12.3 Case study for Upgradation of Wharf

Now, let us take a closer look at some of the most typical circumstances where sheet piles are used for upgradation of wharves. Here, three typical cases for upgradation of existing wharves or quay structure with distinct nature are discussed here;

i) Upgradation of wharf with gravity retaining wall
ii) Upgradation of quay wall with RCC diaphragm wall, and
iii) Upgradation of open berthing structure with revetment

12.3.1 The Fifth Dock at Port of Antwerp

It’s a case study for upgradation of wharf with masonry type gravity retaining structure. At the Port of Antwerp, the Fifth Dock wall was constructed in early nineteenth century to draw the ships of draft less than

7 meter. Later on, existing dock wall needed to deepen to allow cape size vessels for mooring. In 1994, the existing dock wall was upgraded to meet the demand. The schematic arrangement for upgrade is illustrated in **Figure 12.2**. where, the existing masonry type gravity quay wall is maintained. For deepening and augmentation of structure, new cut-off wall was introduced to safely retain the soil at founding strata. After assessment of geotechnical conditions and their variability, open combi-wall with infill sheet piles of limited length at level of scour protection are installed.

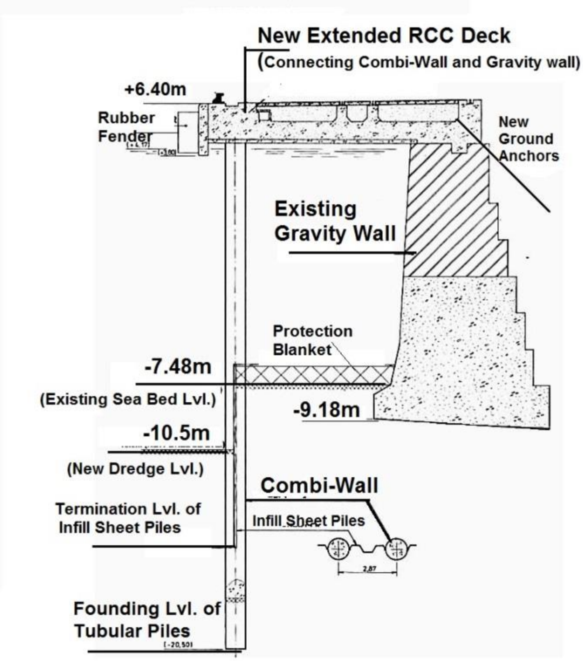

New Extended RCC Deck
(Connecting Combi-Wall and Gravity wall)
+6.40m
Rubber Fender
(+4.17)
(+3.60)
New Ground Anchors
Existing Gravity Wall
Protection Blanket
-7.48m
(Existing Sea Bed Lvl.)
-9.18m
-10.5m
(New Dredge Lvl.)
Combi-Wall
Termination Lvl. of Infill Sheet Piles
Infill Sheet Piles
2,87
Founding Lvl. of Tubular Piles
(-20,50)

Figure 12.2 Typical cross-section of Quay Wall Renovation at Antwerp

New grout anchors are installed to reduce the horizontal forces acting on the wall from earth pressure and increased bollard force, and to reduce eccentricity and inclination of the resulting force at the foundation of the wall. This type of solution for revamping of wharf is more demanding and still less expensive, less time-consuming than other solutions for disregarding the existing quay wall.

12.3.2 Augmentation of General Cargo Berths at Paradip Port

It’s a case study for upgradation of quay structure with RCC diaphragm wall. The existing General Cargo Berths at Paradip Port were more than three decades old consists of 1.10M thick anchored diaphragm wall. The wharf structure was capable to handle the panamax size vessel and was unable to take the cape size vessels being proposed at the Port. Then existing quay wall was planned to reconfigured and deepen to accommodate (larger vessels) up to 1, 80,000 DWT ship size and plan was made to deepen the berthing basin from -11.5m level to -19.8m in future. The comprehensive study of existing berthing structures and sub-soil condition of port area was carried out by the author for planning, design-verification and reconfigure the existing structure to meet the new requirements satisfactorily and assisted the site team on regular basis to ensure the compliance of design requirements. The approved method for augmentation was to install a cut-off wall in front of existing quay wall consisting tube and infill sheet piles (combi-wall) as schematically shown in **Figure-12.3**.

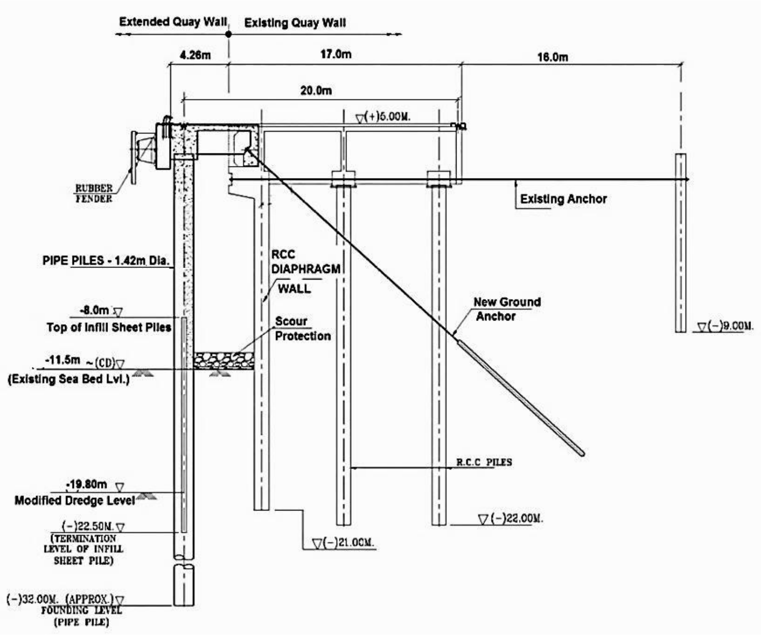

Extended Quay Wall
Existing Quay Wall
4.26m
17.0m
16.0m
20.0m
▽(+)5.00M.
RUBBER FENDER
Existing Anchor
PIPE PILES - 1.42m Dia.
RCC DIAPHRAGM WALL
-8.0m ▽
Top of Infill Sheet Piles
Scour Protection
New Ground Anchor
▽(-)9.00M.
-11.5m ~(CD)▽
(Existing Sea Bed Lvl.)
R.C.C PILES
-19.80m ▽
Modified Dredge Level
(-)22.50M.▽
(TERMINATION LEVEL OF INFILL SHEET PILE)
▽(-)22.00M.
▽(-)21.00M.
(-)32.00M. (APPROX.)▽
FOUNDING LEVEL
(PIPE PILE)

Figure 12.3 Typical cross-section of Augmented Quay Wall

The augmentation process necessarily involved following steps:

i) New cut-off wall configured by deep tubular piles and infill sheet piles. Deep tubular piles functional as load bearing piles cum main retaining wall and infill sheet piles protect the soil at the interface to permit the dredging upto -19.80M.

ii) Tie-backs were installed at each mooring location and intermediate point of berth to accommodate the additional lateral loads due to handling of larger ship sizes.

iii) The axis of the combi-sheet pile wall is aligned approximately 3.26 m away from the face to widen the berth deck required for berth widening and mechanization.

The proposed construction scheme was economical, saved considerable time for the project and it is a very successful first tubular combi-wall application in India and presently under operation.

12.3.3 Upgradation of Port Elizabeth marine terminal

It's a case study for upgradation of wharf with Open berthing structure and revetment. Port Elizabeth marine terminal is located near New York City. The existing wharf structure at Berths 82 through 98 was constructed in early 1970's to serve as general cargo berths. The berths were suitable to handle the Panamax size vessels. Due to demand of containerised cargo, terminal was made equipped to handle the container vessels. Later on, the Port Authority decided to upgrade the thirty-year-old wharf structures to handle the next generation of container vessels by increasing the draft of harbour. Plan was made to deepen the berthing basin from -10m level to -15.20m level to suit the 1,50,000 DWT vessels. The original berthing platform was made of reinforced concrete supported on timber piles. To provide lateral stability to the mooring cum berthing system, pile head were anchored through tie rods and anchor piles.

The deepening and improvement process of wharf involves the followings;

a) Installation of a Combi-sheet piles (combination of H-beams and Z-piles) as cut-off wall along the face of the wharf to enable the water depth to be increased to -15.2 m.

b) Rehabilitation of structural deck element to support the larger container handling cranes and replacement of existing fenders, capable to resisting the berthing energy of proposed 150,000-DWT ships,

c) Installation of additional anchors with deadman to minimise the deflection and stress in berthing structure against the mooring forces.

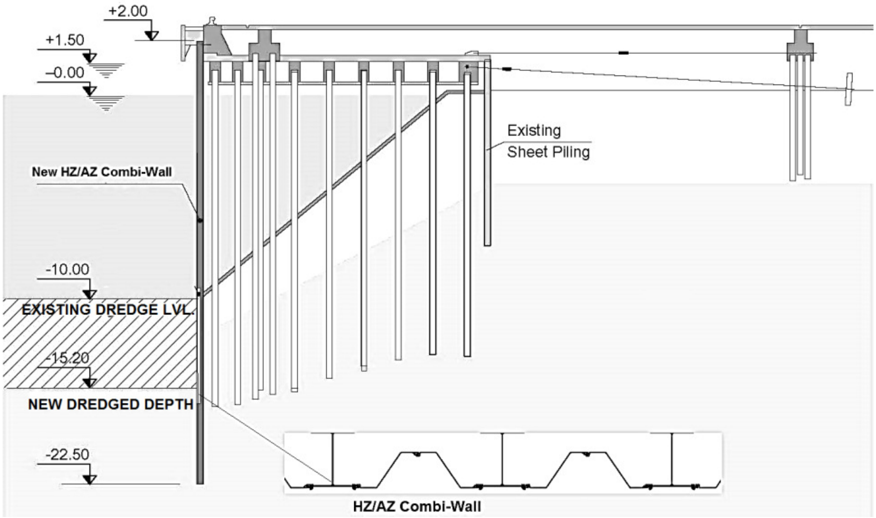

+2.00
+1.50
–0.00
New HZ/AZ Combi-Wall
Existing
Sheet Piling
-10.00
EXISTING DREDGE LVL.
-15.20
NEW DREDGED DEPTH
-22.50
HZ/AZ Combi-Wall

Figure 12.4 Typical cross-section of Modified Wharf at Elizabeth Terminal

The advantage of the cut-off sheet pile wall chosen is the ability to retain the existing founding stratum beneath the wharf while permitting the area in front of the toe wall to be dredged to the desired depth. The primary or H-beams were installed along the berthing line from wharf deck and intermediate short Z- piles were installed between H-beams using follower piles. It was a very cost-effective solution for the renovation of wharf.

12.4 Challenges in Design for Upgradation

Upgradation of quay structure means - ***Bridging the gap between past and the present for future requirements***. This could be achieved by "upgrading, strengthening and extending lifetime". The major challenge for design and renovation of such projects are to assess the state of existing structures and formulation of suitable plan for reconfiguring, deepening and strengthening of structure complying the prevailing design code provisions. The procedures generally followed are outlined below;

12.4.1 Assessment of Geotechnical Condition of Site

For this purpose, a well-structured and comprehensive geotechnical report is necessary for quay wall development, as the forces, load bearing resistance and characteristics of deformation by the various types of soil types can be very different. For every civil engineering design, it is important to have a thorough understanding of the sub-soil conditions. It must quantify the state of different layers of backfill, condition of founding and bearing strata and passive resistance to be offered below the mud line after dredging. These data should be collected for entire stretch of structure to be augmented. Based on above assessment, the effect on structure due to deepening of basin and dredging of soil shall be assessed for short term and long term effect.

Past investigation reports, of existing wharf are also very important. The comparison of new sub-soil investigation and past existing report may demonstrate soil disturbance due to the execution of the existing infrastructure and dredging works and should be accounted for in the design of the upgrading works. Furthermore, the selections of suitable material for modifications of structure and construction techniques to be implemented are largely dependent on the type of existing soil stratum. The assessment of the geotechnical conditions and of the state of the existing infrastructure is vital to evaluate its ability to support the additional loads, to be included as an active element in the upgraded wall.

Therefore, the soil investigation should preferably be performed on the platform and at sea-side of the quay to obtain comprehensive information of the sub-soil conditions. And, finally it is important to note that the soil investigation program should not be designed for the purpose of the final conditions only to evaluate the overall stability of the system but also to determine the most reliable construction method and evaluate the construction risks.

12.4.2 Assessment of the Condition of Structure

A study of the past available data of the concerned structure provides insight into the design condition and the construction technique employed for the structure. Nowadays the most frequently used materials for quay wall construction are high grade concrete and reinforcement steel and steel sheet piles of different geometries. However, the old construction may have relatively lower grade of concrete, reinforcement steel and also rubble masonry wall. Therefore, the information of construction materials used in the structure is very important. Further, the existing condition of material considering the ageing effects and environmental degradation must be evaluated within the design process.

The condition survey shall include the physical survey employing established methods like NDT tests, partial destructive tests etc. The state of existing structure shall be evaluated, which may include the followings;

- The deviations in design and actual construction of structure, or list of other modifications, maintenance carried out during operations
- Wear and tear happened during actual operation or the conditions due to unusual loadings,
- Sign of distress, if any visible.
- The effect of creep, ageing and environment on structure

For condition survey of existing structures, several techniques have been developed to assess the conditions of existing concrete or steel elements. However, in quay structure most of its structural elements are in buried condition or under water and the inherent limitations of these NDT techniques should be well acknowledged. Very often a large part of uncertainty remains despite the efforts to scrutinise the existing construction. Therefore, the results should be judiciously corroborated by employing various techniques of non-destructive tests.

12.4.3 Compliance of the structure with present Design codes

In last few decades, the design codes have undergone significant evolutions, upgradations and changes. More stringent load assessment and durability criteria have been introduced. So, the design which was valid originally, might appear insufficient in regard with the present codes. Therefore, the upgradation of existing structure cannot be limited to some refurbishment but strengthening and stabilisation of existing structure before makeover changes.

12.5 Methodology for Deepening and Strengthening

Deepening of basin in front of an existing quay wall to cater larger vessels means existing quay wall needs to support;

- → Deep soil retention due to dredging in front of wall,
- → Larger loads from ships and cranes,
- → Reduction in embedment depth of wall after dredging

Therefore, setting of functional and technical requirements to upgrade the existing quay structure in terms of operation, loads, basin depth and design methodology defining serviceability and ultimate limit states as well as design life is very important.

12.5.1 Setting of Functional and Technical Requirements

The modified quay structure must fulfil the following basic functional and technical requirements:

a) Retaining function

The structure has to safely retain soil and water (if required) after deepening of port basin. Typical for this aspect, the retaining height for structure must be agreed for the established dredge depth.

b) Bearing function

The quay wall must safely sustain the new additional imposed loads by cranes, vehicles and the stored goods. It is essential that the handling equipment for the goods is safe. Due to the requirement for high handling speeds, extra horizontal forces may be generated due to acceleration of the cranes. In some cases, additional storage areas and transhipment facility needs to be incorporated in the design to meet the functional requirements.

c) Mooring function

The structure must be able to ensure that the larger proposed ships can be safely moored at their berths.

12.6 Options for upgrading of wharf

The methodology for upgradation of quay structure shall essentially involve a robust design and construction method. While deepening of basin, it shall account the technique for integrating and strengthening of existing structure, avoiding it being affected adversely by the construction activities.

The typical solutions for upgrading the wharf or quay structure are;

12.6.1 Construction of additional new cut-off wall

Construction of an additional new structure or cut-off wall in front of the existing quay wall provides a suitable retaining and protection system. Use of an open combi-wall with infill sheet piles as cut-off wall may provide a suitable retaining or protection system as well as load bearing member for new imposed loads. Large modulus piles can efficiently bear the loads in both vertical and horizontal directions according to the system requirements. The additional new structure or cut-off wall in front of the existing quay should be suitably integrated with existing structure to form a composite system.

12.6.2 Strengthening of Existing structure

i) Installation of Tie-Backs

To strengthen the existing structure, installation of additional ties or new grout anchors reduce the horizontal forces acting on the wall from earth pressure and increased bollard force is an essence. They also reduce the eccentricity, lateral deformation of wall and provide the stability to the foundation. The anchor can be positioned horizontally or inclined. In the latter case the anchor initiates a vertical load on the wall. If pre-stressed ground anchors are carefully installed, minimum horizontal deflection will occur in the wall, thus prevents the subsidence in backfilled area. After installation of series of pre-stressed ground anchors, the whole backfilled area behind the quay wall changed into a compression zone, and behaves like a massive gravity wall.

It must be noted that, the dredging or excavation in front of existing wall causes to change in stresses in the ground enveloping the structure. These stress change causes extra lateral earth pressure on wall. As free-standing height of wall or pile increases due to deepening of soil, the introduction of tie backs or anchorages control the deformation in wall and provide supports for stabilisation.

ii) Strengthening of Back-fill

If more strengthening or stabilisation of existing ground around the wall is required, various techniques of ground strengthening like soil nailing, vibro-floatation etc. may be used. However, the anchored wall with grouted anchors or tie-back arrangements exhibits more stabilized behaviour for deepening of quay wall.

Above upgradation technique should be employed, when the geotechnical conditions at the site are conducive for installation of new structural elements. The construction methodology should cause minimum impacts on existing structure and surrounding environment. Monitoring schemes should also be set to mitigate the unexpected and detrimental effect on existing structure. Application of Sheet Piles in

12.7 Application of sheet Pile in Marine Bulkheads

Sheet piling can be used a number of ways in the design and construction of anchored wall system. An anchored sheet pile bulkhead provides a vertical wall, behind which useable land area may be created and in front of which shipping may tie up for loading and unloading.

It may be located parallel to shore (marginal wall) or perpendicular to shore (finger pier). Anchored bulkheads may be built for planned dredged depths of up to about 20 meters of water or more by utilizing the stronger sections, combined with high strength grades of steel. Sheet pile walls are also utilized in the construction of deck-type wharves, as a screen (retaining) wall between the rear of the deck and the land area.

Anchored sheet pile is popular pick for these types of construction. It offers large number of configurations and solution to troubleshoot any retaining wall situation and scenarios or sub-soil conditions. The anchored wall transfers loads due to retained soil and live load surcharge to elsewhere in the ground mass and by the wall and its support system. How this transfer occurs depends upon the type of wall, the in-situ stress state, strength and stiffness of ground, the wall and its support system, and also on the method and sequence by which the wall and support system are constructed. The design of anchored bulkhead requires a holistic approach accounting the geography of the site and geotechnical parameters of the sub-soil. This requires a full understanding of the role of the wall alongwith overall structure and how it interacts with its support system. Special high modulus sheet pile wall systems have become a key factor in deep-water marine facilities construction. This method utilizes large “king piles” as the main strength units. Wall unit strengths several times that available with conventional sheet piling sections. Furthermore, it shall be noted that major economies are possible at the scheme design stage by reviewing the natural ground formation, construction method, selection of anchor and type of wall to be used. Some of the economical and efficient schemes of bulkhead construction based on geographical formation of the site are discussed here;

12.7.1 Construction of Bulkhead in Land Mode

On suitable ground condition, embedded sheet pile wall and anchorage system can be installed prior to dredging or excavation. In land-mode construction, the sheet pile has to be driven first into the ground and after installation of anchors, dredging can be made in front of the wall. The construction of wall in land mode is fast and convenient for handling of sheet piles and movement of cranes, templates etc. The activity required for earth work and backfilling is also minimised. Anchored Wall with Single Level of Anchorage is the very common for marine bulkhead. Generally, tie-rods are installed above the mean water line. Care must be exercised to see that the anchor wall or deadman arrangement does not settle after construction. This is generally not a problem in undisturbed soils, however, where the anchorage must be located in unconsolidated fill, piles may be needed to support the blocks. Also, the soil within the passive wedge of the anchor should be compacted to at least 95 per cent of maximum density unless the deadman is forced against firm natural soil. For an anchorage system to be effective, it must be located outside the potential active failure zone developed behind a sheet pile wall. If it is located in unstable ground or in the active failure zone, anchorage capacity may be impaired, which shall be essentially avoided.

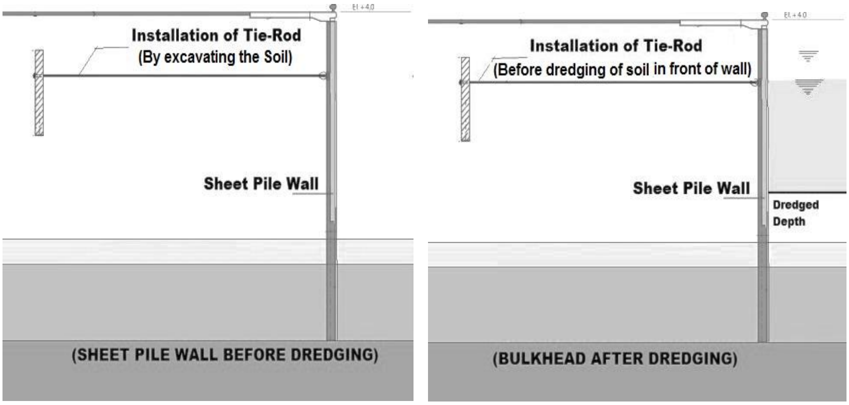

El +4.0
Installation of Tie-Rod
(By excavating the Soil)
Sheet Pile Wall
(SHEET PILE WALL BEFORE DREDGING)
El +4.0
Installation of Tie-Rod
(Before dredging of soil in front of wall)
Sheet Pile Wall
Dredged
Depth
(BULKHEAD AFTER DREDGING)

Figure 12.5 Schematic Details of Bulkhead Construction in Land-Mode

Where, the anchorage must be located in unconsolidated fill, additional piles may be needed to support the wailings and anchor wall. In such cases longer sheet pile at certain interval shall be driven in the alignment of anchor wall, which will act as bearing piles for wailings to avoid settlement. Vertical supports or encasement for tie-rod shall be introduced to avoid excessive sag in tie-rods. A schematic detail of land mode construction of bulkhead supported by consolidated or firm soil is shown in **Figure-12.5**.

After installation of wall, the dredging or an excavation of ground in front of wall causes minor deformation and redistribution of stress in the wall and associated ground. These changes cause to loosening of soil behind the retaining walls to exert lateral earth pressure on wall, which must be accounted according to the state of mobilisation of soil.

12.7.2 Construction of Bulkhead on Unconsolidated Ground

If ground formation comprises of very loose or compressible strata upto a limited depth, the existing soft soil strata needs to be dredged and replaced by compacted granular fill prior to the construction to avoid unnecessary settlement of anchorage system. If complete replacement of unconsolidated backfill is not possible, anchor pull may be transferred through rigid pile formations founded in firm strata.

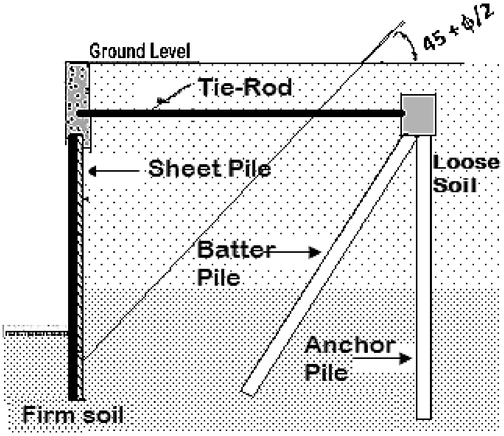


Figure 12.6 Schematic Details of Piled anchor

Application of anchor pull in piles typically involves significant lateral loads and/or moments, resulting in significant internal bending moments and shear forces. **Fig 12.6** Shows, tie rods are supported by group of piles. These types of configuration, often referred to as “A-frame,” which resist the lateral load from an anchored wall through axial compression, tension, and shear forces through the piles.

Vertical piles are commonly used to resist vertical loads because they are easier and less costly to construct; however, batter piles typically provide the necessary resistance with less displacement and often with a smaller group of piles. The anchor pull force exerted by tie rod tends to induce compression in front pile and tension in rear pile. Application of relieving platform is a very effective in reducing the lateral earth pressure on wall triggered due to surcharge loads. Relieving platform reduces the active earth pressure on the retaining wall, which leads to reduction of bending moments, thus a reduction of the profile (section modulus) and sheet pile lengths are possible. Further, it provides an operating platform for crane, traffic loads and storage of goods.

12.7.3 Bulkhead with Rigid Portal

The anchor loads of a bulkhead can be supported by rigid pile frame known as A-frame as discussed in section 12.7.3. Instead of A-frame, it can be resisted by portal frame. If there is space constraint to accommodate the complete anchorage system behind the wall, portal action may be used for the construction of bulkhead. The schematic detail of wall is shown in **Figure-12.7**.

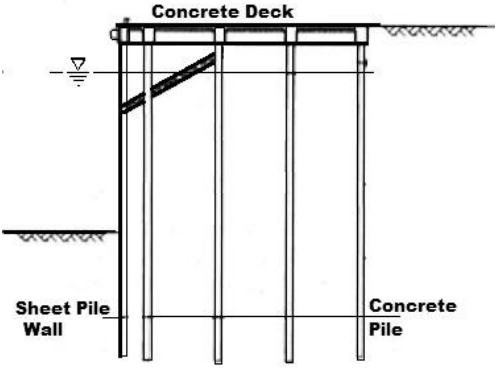


Figure 12.7 Bulkhead with Portal Frames

It is a combination of rigid vertical piles and beams/slabs in the form of space frame. This type of structural system is suitable to support the deck slab for a relieving platform as well as a heavy operating platform behind the wall. The retaining wall or front sheet pile wall shall be judiciously organised to relieve the wall from hydrostatic pressure and minimise the lateral earth pressures.

12.7.4 Construction of Bulkhead at Sloping Ground

If shoreline topography has steep slope and sea bed is suitable for placing the backfill without causing mudflow or instability to the sea bed; careful planning of work for backfilling and sheet piling can be streamlined to optimise the earth work and total duration of construction of bulkhead. For placement of anchor selected backfill material should be placed. The fill around the anchor unit shall be compacted to at least 95 percent of standard proctor dry density. If soil conditions warrant the consideration of settlement, suitable pile supports may be used to support the tie arrangement.

The construction works may be exercised in following sequence;

i. Backfill shall be placed in two stages. First stage backfill should be placed to form a stable embankment along the proposed anchor wall. Gradation of material should be suitable for prevailing waterfront environments.

ii. After verifying the slope stability of the first stage backfill, anchor wall shall be installed with wailing and tie-rods as schematically shown in **Figure12.8**.

iii. Front wall can also be installed in succession of anchor wall with a suitable lag.

iv. Tie-rods connections between front wall and anchor wall should be provided with turn buckles, without causing a slack or over-tightening. To avoid the sag or vertical deflection in tie-rods, vertical supports at 6m to 10m interval may be provided, if anchor wall is placed far behind the front retaining wall.

v. Second stage backfill between the constructed wall should be placed progressively in layers. The backfilling material should consist of non-cohesive hydraulic fill and the fine contents with a diameter < 0.06mm should be limited to 10 percent at the most. To maintain the fill density according to the stipulated design condition, suitable compaction effort may be employed on hydraulic fill or dumped material. Placement of compacted fill reduces the material compressibility, increases its strength and thereby minimises the post construction settlement behind the bulkhead.

vi. During placement of backfill, heavy compaction equipment shall be kept well back from newly constructed walls. Compaction within 2 meter of the back of the wall shall be accomplished with lightweight hand operated compaction equipment.

vii. Finally, the top surface of backfilled shall be capped with non-plastic, fine grained fill to avoid the percolation of surface runoff at backfill.

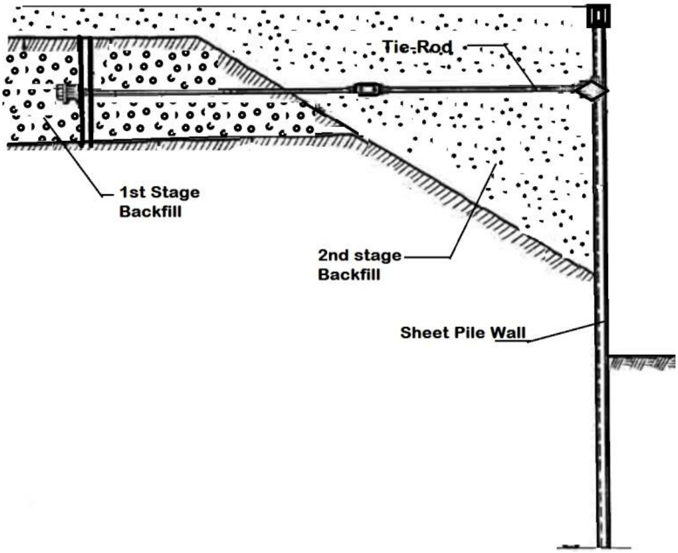


Figure 12.8 Bulkhead Constructions along Steep Sloping Ground

12.7.5 Bulkhead with Plate Anchors

If bulkhead is located in considerable deep water, where deep backfilling is required, the possibility for installation of plate anchor may be explored. The area where dead man or tieback construction is not adaptive due to deep backfilling behind the wall, plate anchor provides a very cost effective and stable alternative.

Plate-Anchors are used to transmit the tensile forces from the structure to the soil which derive the anchorage resistance from the shear strength and dead weight of the soil surrounding the plates. The effectiveness of plate anchors increases with the increase in depth. These may be placed horizontally, or at an inclined position depending on the load orientation or type of structure requiring support. The detail for design of plate anchor is provided in section-7.4. A typical detail of plate anchor installed to construct a quay structure at Hamburg Port, Germany is shown below in **Figure -12.9**.

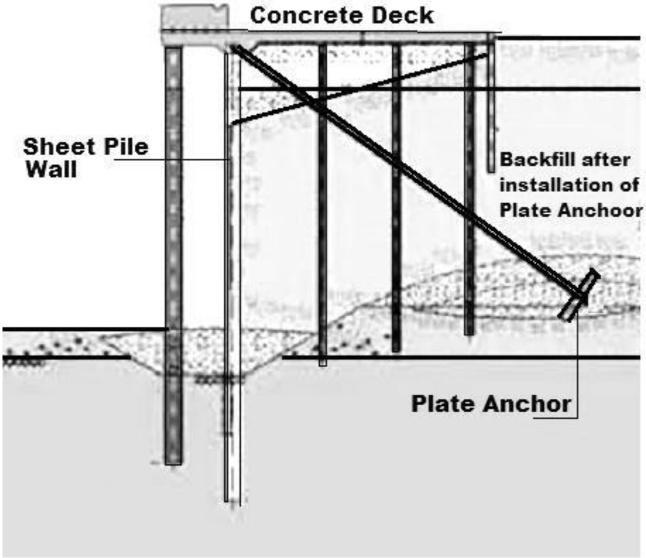


Figure 12.9 Bulkheads with Plate Anchors

The construction bulkhead with plate anchors may be followed in following sequence;

i. Install the front wall in proposed alignment upto the designed depth with temporary supports, if required.

ii. Assemble and Install the plate anchors behind the wall to meet the design requirements. The schematic arrangements as shown in **Figure- 12.9**.

iii. After installation of anchors, suitable backfill materials shall be placed in layers. Initially, the backfill should be placed over the anchors to get it stabilised against the lateral pull and backfilling behind the wall shall be placed with a suitable lag.

iv. If required, minor or suitable compactions may be employed by using deep vibrators to stabilise the backfill.

v. Finally, the fill shall be placed up to the design formation level.

vi. Relieving platform or top surfacing may be placed according to the design requirements.

If soft strata exist at sea bed level, it shall be removed prior to the installation of anchor and placing the fill. For effectiveness of Plate anchors, it should rest on non compressible strata.

12.7.6 Construction of Wall with Rock Anchors

Rock anchors or ground anchors may be installed to facilitate the anchorage to the retaining wall, if limited backfill is to be placed behind the wall and rock strata is available at considerable depth below the existing ground level. An anchored bulkhead with ground anchors creates an internally stable soil mass to resist the failures and deformations in wall. The anchors provide the required stabilizing forces, which are transmitted back into the ground. The anchor bond length should extend into the ground having stable soil or rock mass, which does not envelope any potentially critical failure surfaces behind the wall.

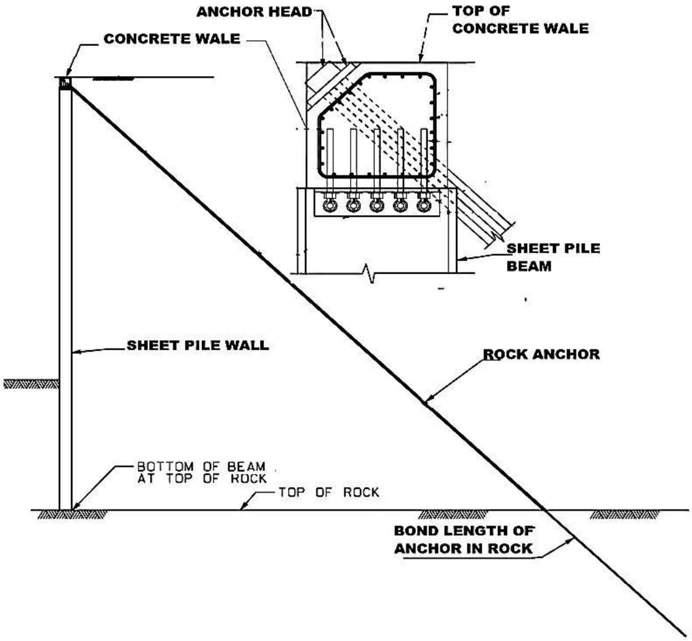

ANCHOR HEAD
CONCRETE WALE
TOP OF CONCRETE WALE
SHEET PILE BEAM
SHEET PILE WALL
ROCK ANCHOR
BOTTOM OF BEAM AT TOP OF ROCK
TOP OF ROCK
BOND LENGTH OF ANCHOR IN ROCK

Figure 12.10 Bulkheads with Rock Anchors

The construction of bulkhead with rock anchors may be followed in following sequence;

i. Install the main wall up to the designed depth in proposed alignment,

ii. Place the compacted backfill behind the wall, keeping the wall in vertical cantilever position. The structural strength of wall and its embedment length into the ground shall be safe against the applicable loads during the construction stage (as a cantilever wall with limited surcharge) and at final stage (as anchored wall) complying all the operating loads.

iii. Install the rock anchors or ground anchors behind the wall to provide lateral stability against the backfill. Series of anchors may be placed at each king pile locations as shown in **Figure-12.10**.

iv. After construction of capping beam for the wall, final backfill shall be placed.

v. If consolidation settlement is expected behind the wall due to presence of compressible sub-soil strata and application of load surcharge, relieving platform may be used.

12.7.7 Bulkhead with Multiple Anchors

Sheet piles resist soil and water pressures by functioning as a beam spanning vertically between points of support. Anchored bulkheads obtain their stability from the support provided by penetration into the soil below the dredged bottom combined with an anchorage support system installed near the top of the wall. In case of deep retaining wall, the span length of sheet piles become very large which results in larger bending moments and shear forces in sheet piles.

By introducing multiple anchors at various positions in the wall as shown in **Figure-12.11**, the span length of vertical pile reduces to optimise the section modulus and embedment depth of pile. Use of multiple anchors in bulkhead, increases the wall stability and enables to build a taller wall using lighter sections. It can be effectively installed, if variation in flood level or tide level at water front site is very large.

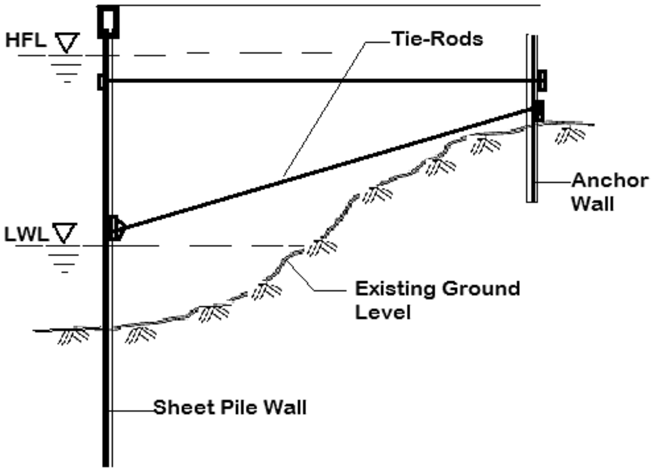


Figure 12.11 Bulkhead with Multiple Anchors

12.8 Characteristics of Backfill for Bulkhead

Backfill material for construction of a bulkhead should be permeable, typically clean granular type. Physical and mechanical properties of backfill or reclaimed material should be determined by laboratory testing before use. Compaction of dumped backfill shall be carried out to reduce the materials compressibility, increase its strength and reduce the post construction settlement and should agree the shear strength and density index adopted in design. Therefore, selection of fill and compaction shall be carefully stipulated both in the degree of densification and the method of attainment.

12.8.1 Effects of Backfill Extent

During design the retaining wall, the design loads based on active pressure, must be taken into consideration upto the extent of the backfill. To anticipate the lateral earth pressures correctly, the construction methodology and wall/soil placement technique, the direction of wall movement relative to the soil, and the prevailing groundwater conditions should also be accounted.

The maximum active pressure may develop either from limiting conditions occurring entirely within the backfill material or from potential failure surfaces within the material present behind the zone of backfill. Therefore, to anticipate the lateral earth pressures correctly, the designer should anticipate the construction methodology and wall/soil placement technique, the direction of wall movement relative to the soil, and the probable worst groundwater conditions. In general, the designer should have full overview of the structure.

12.9 Assessment of Overall Stability

To analyse the bulkhead structure and calculate the forces in the various parts of it with reasonable accuracy, it is necessary to represent the structure in a simple geometric model. Virtual geometric model of structure should be based on prevailing sub-soil parameters. The lateral (active) earth pressure and soil subgrade modulus behind the wall should be evaluated in accordance to slip wedge line based on Mohr's rupture envelope through backfill and loadings for following cases;

a) Construction stages,
b) Short term or initial service loads and
c) Long term service loads

If bulkhead structure derives their stability from an anchor, it should be treated as integral part of the system. The details for design of various types of anchors are given in chapter-7. Anchors should be placed against firm soil and its geotechnical and structural stability should not be impaired by the construction activity. A minimum safety factor of 2 is recommended for all anchors and anchor blocks.

The verification of the overall stability of bulkhead structure must account the possible unfavourable state of deep sliding planes, stability of the anchors, heaving of the stabilizing soil layers on the passive side of sheet pile, condition of piping and local buckling in steel sheet pile wall. A local equilibrium checks should also ensure that forces transmitted to the ground do not cause local failure of the soil.

In waterfront area, structures are subject to the effects of tidal variations and storm runoff. The design loads should take account of the possible build-up of ground water levels behind the wall. To assess the design level of water table, natural variations in the ground water table, and the provision of effective drainage and the drainage characteristics of backfill should be considered.

12.10 Development of Design Report

The design report of bulkhead and retaining structures should incorporate the applicable load combinations arise due to environmental situations, operational requirements in association with geophysical and structural stability. The stability assessment report for bulkhead structure should generally contain the followings:

- Layout of the structure with all essential planned dimensions,
- characteristic values of all loads and actions,

- Sub-soil strata and its geotechnical parameters which are under the influence of bulkhead,
- variation in water levels on land side and water side,
- combinations of load cases applicable according to the specified design standards or codes,
- proposed construction materials and their design resistance values,
- description of the intended design and stability verification procedures,
- information or description of the applicable design codes, standards, guidelines and other calculation aids,

Finally, existing ground along the alignment of bulkhead should safely sustain the imposed loads due to reclaimed fill, structural elements, operational loads and environments. The total post construction settlement of the fill shall also be within specified limits.

Glossary

Actual Wall Length It is the shortest possible length for a specified sheet pile wall. Actual length of the wall is defined as (panel width x total number of panels).

Allowable Bending Stress Flexure, or bending, is an indirect stress due to moments in the beam caused by the beam's loads and reactions. In general, the accepted engineering practice usually accounts for a safety factor for the allowable bending stress of 66% of yield stress or 0.66 x fy.

Allowable bearing pressure In working stress design, it is the maximum pressure that may be applied to a soil or rock by the foundation unit considered in design under expected loading and subsurface conditions towards achieving desired performance of the foundation system. In limit stress design, allowable bearing pressure commonly corresponds to serviceability limit states for settlement not exceeding 25 mm towards achieving desired performance of the foundation.

Anchors Anchors as the general expression used to describe the anchoring system at the back of a retaining wall, such as deadman anchors, anchor plates (or anchor screens), screw anchors, ground anchors, anchor piles and expanded bodies. It is a device or structure which, by interacting with the soil or rock, generates the required anchor force. Anchors act a support to restrict the lateral deflections of wall.

Anchorage A mechanical assemblage consisting of wales, tie rods, and anchors which supplement soil support for an anchored wall.

Anchor force The reaction force (usually expressed per meter length of wall) which the anchor must provide to the wall.

Anchored wall A sheet pile wall which derives its support from a combination of interaction with the surrounding soil and one (or more) mechanical devices which inhibit motion at an isolated point(s).

Backfill A generic term applied to the material on the retained side of the wall.

Bearing or contact pressure The pressure applied to a soil or rock by a foundation unit.

Bearing capacity or geotechnical capacity The maximum or ultimate soil resistance mobilized by a loaded foundation unit, e.g., a footing, or a pile. (The structural capacity of a foundation unit is the ultimate resistance of the unit itself as based on the strength of the building materials).

Bearing piles Bearing piles are structural elements (steel H-piles or steel pipe piles) incorporated into the foundations of civil engineering works. They are used for resisting axial compressive or tensile forces, moments, and transverse (shear) forces. The vertical bearing resistance is achieved by base resistance, shaft friction or a combination of both. In other words, it is a post or pile which carries the weight of a foundation, transmitting the load of a structure to the bedrock or competent subsoil strata without detrimental settlement.

Bending moment capacity Bending moment capacity incorporates the strength of the section due to its geometry (section modulus) and takes into account the steel grade: Bending moment capacity = elastic section modulus [m^3/m] × minimum yield strength [kN/m^2] (without safety factor)

Blow Count The number of hammer impacts or blows required to move the pile a given unit of measurement - most commonly 1 foot (also could be 1 inch or other increment).

Bracing Struts perpendicular or at an angle to the front face of a retaining wall that support the wall and are usually connected to the wailings is defines as bracing.

Cantilever Wall Wall whose stability depends solely upon the penetration of the sheet piling into the ground is defined as a cantilever wall.

Cap block Cast steel insert between the hammer and the helmet that houses the hammer cushion.

Cellular Cofferdam Cellular cofferdams are gravity structure, constructed of straight web profiles with interlock tensile strength sufficient to resist the circumferential tension developed in the cellular walls due to the radial pressure of the contained fill. The stability of these cells is obtained by the self-weight of the fill.

Circular cells This type of cofferdam consists of individual cells of large diameter connected together by arcs of smaller diameter.

Diaphragm cells This type of cofferdam consists of two rows of circular arcs connected together by diaphragms perpendicular to the axis of the cofferdam.

Coating Area Coating area is simply the required area for anti-corrosion protective coating to be applied on a steel pipe, beam or sheet. Usually measured in square feet or square meters.

For Z and U sheet piling: the surface of the Z-profile section coated, excluding the ball and the interior of the socket.

For combined wall systems: the surface of the combined wall system coated, including the facing surfaces of the king pile (beams or pipes), intermediate sheet piling, and connectors.

Combined Sheet Piling Walls Combined sheet piling walls are comprised of primary elements also known as “king piles” and secondary elements. Primary elements are most often O-Pile systems (pipes with welded connecting sections) or H-sheet piles (either beams with flanges or beams with welded connection sections). Secondary elements are most often Z or U-type sheet piles and/or connector piling sections in various configurations.

Combined Sheet Piles: H + Z combined sheet piles A combined steel sheet piling wall system comprising H-sheet piles connected to Z-sheet piles by special sheet piling connector sections to form a continuous wall.

Combined Sheet Piles: O + Z Combined sheet piles A combined steel sheet piling wall system comprising O-shaped pipes or tubes, acting as the primary elements, and connected to Z profiles via special sheet piling connecting sections.

Connector Connectors are modular piling section that connects two sheet piling sections together. Used either when needing to turn a corner, or for a transition from one type of interlock to another. It is also widely used to form all types of combined sheet pile walls, including pipe pile retaining system.

Contiguous wall Contiguous sheet pile walls are a series of steel panels that vertically interlock to form a continuous wall. Contiguous sheet pile walls may provide an efficient, economical and time-saving solution in certain types of applications, such as cofferdams, cut and cover tunnels, retaining walls, seawalls and containment walls, if large section force and moments are not expected.

Contiguous O-pile An O-shaped sheet pile is a pipe section with attached connector sections so that one section can be driven into the next to form a continuous steel wall with the same load bearing element.

Contiguous U-sheet Pile A U-shaped sheet pile is comprised of a web with two flanges which form a shape similar to the letter U. The interlocks of a single u sheet pile are positioned at both flange ends and are located on the wall axis.

Contiguous Z-sheet Pile A Z-shaped sheet pile is comprised of two flanges having substantially parallel outer faces and an oblique web connected to the two flanges which form a shape similar to the letter Z. The interlocks of a z-sheet pile are positioned beside the wall axis located at the flange ends and are interlocked into on another to form a continuous steel wall with the same element.

Corrosion Resistance Corrosion resistance is the percentage of the bending moment capacity remaining in a steel section after a given period of time, according to corrosion data stipulated in relevant standards.

Depth Distance from front to back of the wall is known depth of wall.

Design Life The amount of time for which a steel retaining wall is designed to last. Steel sheet piling is usually classified into two construction applications: temporary and permanent.

Design Moment Design Moment is the maximum bending moment from the static calculation of the sheet pile section.

Design Strength Design strength of a steel piling structure usually factors into consideration the maximum bending moment for the static calculation of the sheet pile section under given loads as well as the steel grade of the sheet piling elements.

Double U-pile Two numbers of single U sheet piles threaded together with crimp or weld along the common interlock allowing for shear force transmission is known as double U-pile.

Dredge line A generic term applied to the soil surface on the dredge side of a retaining or floodwall.

Dredge side A generic term referring to the side of a retaining wall with the lower soil surface elevation or to the side of a floodwall with the lower water elevation.

Driveability The ability of a sheet pile or bearing pile to be driven through the ground strata to the required penetration depth without detrimental effects is known as drivability.

Driving Ratio Driving ratio is the percentage of the wall length that will need to be driven to the full design depth (king piles). The remaining percentage will be easier to install because of the shorter intermediate sheet piling.

The lower the driving ratio, the easier and quicker it will be to install the system. Driving ratio is calculated by the following formula:

D = (Wp − Wi) / Wp
Wp = panel width
Wi = intermediate sheet piling width (with connectors)

Fillet Radius In hot rolled section, the slightly curved portion where the transition between web and flange happens is called fillet. The radius of the fillet is simply called fillet radius.

Flange Width The top and bottom horizontal plate, like sections of an I-beam, are called flanges. Flange width is simply the width of the flanges.

Flexibility It is the amount of swivel at each wall component interlock (measured in degrees or radians). Greater flexibility allows for easier driving in difficult soil conditions. A combined sheet piling wall systems flexibility usually allows for easy setting of the intermediate sheet piling between the already driven king piles (beams or pipes).

Followers It is an extension, usually steel beam or sheet pile, used between the top of the pile and the hammer to extend the driving range of the pile (usually below water or below ground).

High Modulus Wall High modulus wall are high-strength retaining wall formed by interlocking steel elements that have the same geometry. The elements may consist of fabricated profiles to obtain a high section modulus.

Indicator Pile Piles that are driven in advance of the permanent piles for the purpose of determining the length of foundation piles by dynamic load testing are known as indicator piles. Indicator piles differ from test piles in that static load test is not necessary on an indicator pile.

Interlock The portion of a steel sheet pile or other sheeting that connects adjacent elements by means of a thumb and finger or similar configuration to make a continuous wall is known as interlock.

Interlock Strength Maximum pull that a sheet pile interlock can bear is known as interlock strength. Greater interlock strength improves integrity during driving and allows forces to be redistributed laterally along the wall.

Interlock Sealant A steel sheet piling interlock sealant is used to prevent the penetration of water through a steel sheet pile system. Desirable properties of such sealants include insolubility, corrosion resistance, and adhesion.

Jagged Wall Jagged walls are "special sheet pile wall configuration in which the single piles are arranged either to enhance the moment of inertia of the wall or to suit special applications.

Jetting Consists of a pump, supply lines, and one or more jet pipes that use water to displace soil and advance a pile into the ground.

king pile wall King pile wall are consists of vertical piles (king, master or soldier piles) driven at intervals, supporting intermediate horizontal elements (boarding, planks or lagging). The king or master piles may be rolled or welded I-sections, tubular or box sections.

Minimum Penetration Minimum depth below ground surface, scour elevation, or bottom of excavation, to which a pile must be driven.

Moment of Inertia Moment of Inertia is a measure of the stiffness of a shape based on geometry. It is most often used for calculating section modulus and is not typically a stand-alone design criterion in the way that section modulus or Design Moment usually used by engineers.

O-pile An O-shaped sheet pile is a pipe section with attached connector sections.

Obstructions Site inspections in the planning phase should identify any obstructions which interfere with alternative alignments or which may necessitate special construction procedures. These site inspections should be supplemented by information obtained from local agencies to locate underground utilities such as sewers, water lines, power lines, and telephone lines. Removal or relocation of any obstruction must be coordinated. Undiscovered obstructions will likely result in construction delays and additional costs for removal or relocation of the obstruction.

Panel Quantity Number of panels used to build a wall solution.

Panel Width Panel width is usually defined as the measured distance between two specific interlocking points of sheet pile section.

Pile Coupler Pile couplers are mechanical friction sleeve used to lengthen a steel tubular or X-shaped pile.

Pipe Diameter Pipe diameter is simply the length across the widest portion of the face of the pipe.

Pipe Thickness It is the measurement of the cell thickness of a steel pipe section usually expressed in millimetres or inches.

Pile Extension A non-drivable addition to the top of a pile. Concrete piles are extended by build-up. Steel piles are extended by welding new sections.

Pile Splice A splice is a drivable structural connection between two sections of piles. All splices are pre-planned and are defined in the Pile Installation Plan.

Plan Tip Elevation Optimum elevation to which piles must be driven to assure design requirements are met. Elevation should be shown in the plans.

Plumb Pile From the survey term "plumb line," it is a pile driven vertically or true.

Practical refusal Practical refusal is defined as 20 blows per inch for up to three inches with the hammer operating at the highest fuel setting or at a setting defined by the driving criteria and less than 1/4 inch rebound per blow.

Predrilling Drilling a hole to place the pile in is called predrilling. There are three reasons for predrilling: (1) a starter hole, (2) holes drilled through embankment fill material to reach natural ground surface, and (3) when rock or strong soil strata will not permit the piles to penetrate to the required depth.

Primary Elements A combined wall type retaining wall composed of primary and secondary elements. The primary elements are normally steel tubular piles, I-sections or built-up box types, spaced uniformly along the length of the wall. The secondary elements are generally steel sheet piles of various types installed in the spaces between the primary elements and connected to them by interlocks.

Projected Section Modulus Projected section modulus is an estimation of the actual section modulus of a given steel component after time and corrosion effects due to inputted environmental conditions.

Propped Wall Propped walls defined as retaining walls whose stability depends upon penetration of the sheet piling into the ground, and also upon one or more levels of bracing.

RAM Striking part of the hammer. The weight (pounds) of the Ram is a part of the equation for Hammer Energy.

Restrike Striking the pile with the hammer after the end of the initial drive, after a specified period of time. A restrike is used to determine the setup of additional frictional resistance on a pile.

Retaining wall A sheet pile wall (cantilever or anchored) which sustains a difference in soil surface elevation from one side to the other. The change in soil surface elevations may be produced by excavation, dredging, backfilling, or a combination.

Retained side A generic term referring to the side of a retaining wall with the higher soil surface elevation or to the side of a floodwall with the higher water elevation.

Sealant A steel sheet piling interlock sealant is used to prevent the penetration of water through a steel sheet pile system. Desirable properties of steel sheet piling interlock sealants include insolubility, corrosion resistance, and adhesion.

Secondary Elements The combined walls as "retaining walls composed of primary and secondary elements. The primary elements are normally steel tubular piles, I-sections or built up box types, spaced uniformly along the length of the wall. The secondary elements are generally steel sheet piles of various types installed in the spaces between the primary elements and connected to them by interlocks.

Section Area The cross-sectional area of a given geometrical profile.

Section Depth Distance from front to back of the section is known as section depth.

Section Modulus Section modulus is the measurement of the relative stiffness of a cross section of steel piling based solely on its geometry calculated as Moment of Inertia divided by neutral axis. It is expressed in units of volume (e.g. cubic centimetres or cubic inches).

Section Weight Weight per wall panel unit (or piece) for a given profile section.

Section Width Width is measured from interlock to interlock, along the horizontal axis of the steel shape.

Setting Ratio Setting ratio is the percentage of the wall length that does NOT need to be driven to the full design depth.

Safety factor It is factor to modify/ reduce the overall capacity or strength of material or ground used in working stress design. The safety factor is defined as a ratio of maximum available resistance to mobilized resistance or to applied load.

Shaft resistance The resistance mobilized along the shaft (side) of a deep foundation. Upward acting resistance is called as positive shaft resistance and downward acting resistance is called as negative shaft resistance (See also negative skin friction).

Sheet Pile Connector A modular component that connects two sheet piling sections together. It is used to form seamless connections between steel sheet piles, and other support systems, such as H-Beam, I-Beam and Pipes/Tubes.

Sheet Piling Height Sheet piling height is the length of each sheet pile section measured vertically.

Soldier Wall Soldier walls are those walls that consist of vertical piles (king, master or soldier piles) driven at intervals, supporting intermediate horizontal elements (boarding, planks or lagging). The king or master piles may be rolled or welded I-sections, tubular or box sections.

Soil-structure interaction A process for analysing wall/soil systems in which compatibility of soil pressures and structural displacements are enforced. The design and construction of a sheet pile wall is dependent on the type of existing sub-soil strata and location of the project. The alignment of a sheet pile wall may depend on its function. In harbour or port construction, alignment is dictated by the water source or where the wall serves as a tie-in to primary structures such as locks, dams, etc.

In other circumstances, the alignment may be dependent on the configuration of the system such as space requirements for an anchored wall or the necessary right-of-way for navigation system. In such projects, a sincere coordination among geotechnical and structural engineers is essentially required from the inception of the project to final placement in operation.

Steel Grade Steel grade identifies the strength value for a type of steel. “Grade 50” steel has a value of 50,000 ksi (Kips per square inch). Designation of steel grade according to EN10027-1 for hot rolled steel sheet pile is denoted as S355GP, S430GP, etc.; where, the demarcation S - denotes structural steel, 355-denotes minimum yield strength in N/mm^2 and GP- denotes for sheet piling. Engineers may also choose a higher steel grade to replace a heavier section with a smaller one if necessary or to fulfil other requests.

Steel Piling Combined Walls Combi-Wall steel sheet pile systems are comprised of two main sections: a steel sheet pile and a “king” pile. The “king” pile may be either a steel beam or pipe pile. Combi walls have proven to be a cost efficient way to achieve high section modulus for retaining wall, cofferdam or dock structures.

Tieback Tiebacks are small diameter grouted anchors, typically inclined from horizontal and used to hold back retaining walls, soldier piles or other structures

Template Used to maintain the pile in the proper location and alignment during driving with swinging or semi-fixed leads. It is constructed of steel and make rigid enough to hold the pile in place.

Test pile An exploratory pile used to determine pile lengths and driving criteria for all production piles. Data from the test pile further provides information on the pile, soil, hammer, and method of installation.

Tie rods Parallel bars or tendons which transfer the anchor force from the anchor to the wales.

Triple U Piling Triple U piling as a sheet pile consisting of three threaded single U sheet piles with two crimped or welded common interlocks allowing for shear force transmission.

U-sheet pile A U-sheet pile is comprised of a web with two flanges which form a shape similar to the letter U. The interlocks of a single u sheet pile are positioned at both flange ends and are located on the wall axis.

U-sheet piles are mostly used as double piles, factory threaded and crimped or welded in the middle interlock for transferring shear forces into the ground.

Waling Wailings are horizontal beam(s) attached to the wall to transfer the anchor force from the tie rods to the sheet piling. These are heavy steel elements in a low steel grade constructed of two U-channels with stiffeners placed orthogonal to the sheet pile.

Wall Length Wall length is simply the overall distance of the retaining, cofferdam or dock sheet pile wall.

Waterproofing A steel sheet piling interlock sealant is used to prevent the penetration of water through a steel sheet pile system. Desirable properties of steel sheet piling interlock sealants include insolubility, corrosion resistance, and adhesion.

Web Thickness The vertical section of steel "I" beam is called the web and the thickness of the web is simply called web thickness.

Weight The average weight (per square unit of wall) is usually the most relevant calculation for comparing piling solution costs of the acceptable section modulus.

Weight Per Area The average weight per square unit of wall is the most relevant calculation for comparing piling solution costs of acceptable section modulus. The lowest weight (per square unit of wall) that meets the necessary section modulus may determine the least expensive solution.

Weight ratio The average weight (per square unit of wall) is usually the most relevant calculation for comparing piling solution costs of the acceptable section modulus.

The lowest weight (per square unit of wall) that meets the necessary section modulus will usually determine the less expensive solution.

- 100% Ratio
  This weight assumes a 100% ratio of sheet length to beam (or pipe) length.

- 80% Ratio
  This weight assumes an 80% ratio of sheet length to beam (or pipe) length. The king piles (pipes or beams) are driven deeper,

typically resulting in shorter intermediate sheet piling as the king piles are the load carrying elements of the system.

- 60% Ratio
  This weight assumes a 60% ratio of sheet length to beam (or pipe) length. The king piles (beams or pipes) are driven deeper, typically resulting in shorter intermediate sheet piling as the king piles are the load carrying elements of the system. (Connector length should be the same as the sheets).

Width Width of a sheet of piling is simply the distance across the section. Usually, it is measured from the interlocking point to interlocking point of each specific sheet pile section.
The width of a combined sheet piling wall panel is measured from centreline to centreline of the king piles at each end.
Wider sheets usually lead to reduced installation costs and less handling.

Z-sheet pile A z-shaped sheet pile is comprised of two flanges having substantially parallel outer faces and an oblique web connected to the two flanges which form a shape similar to the letter Z. The interlocks of a z-sheet pile are positioned beside the wall axis located at the flange ends.

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About the Book

The present book is an up-to-date introduction of steel sheet pile related structures, which is one of the popular pick for retaining system and waterfront structures. The discussion covers all the components of a complete sheet pile related structures in one concise volume and includes the factors to be considered in the planning, investigation, design and installation of steel sheet piles of various forms. Solved examples with detailed computations have been included to illustrate the design and application of various components of a sheet pile system. The book is intended to serve as a guide to the practicing engineers in the early part of his/her professional career and the final year engineering students of civil engineering discipline.

Manoj Kumar is a professionally active civil engineer. He graduated in 1997 from M.I.T Muzaffarpur, Bihar and received his post-graduation in 1999, in Computer Aided Design from the University of Roorkee, Roorkee, India (presently I.I.T Roorkee). He has more than 24 years of professional experience involving multi-disciplinary activity in civil engineering and involved in port-based projects of India from initial concept stage to commissioning. His expertise in structure and geotechnics is used in the planning and implementation of various technically demanding port projects for development of wharves, Bulkheads, Bulk Terminals and Container Terminals in difficult soils to provide innovative solutions.